TECHNICAL REPORT

INVESTIGATIONS OF DEEP-SEA SEDIMENT CORES

I. SHEAR STRENGTH, BEARING CAPACITY, AND CONSOLIDATION

ADRIAN F. RICHARDS

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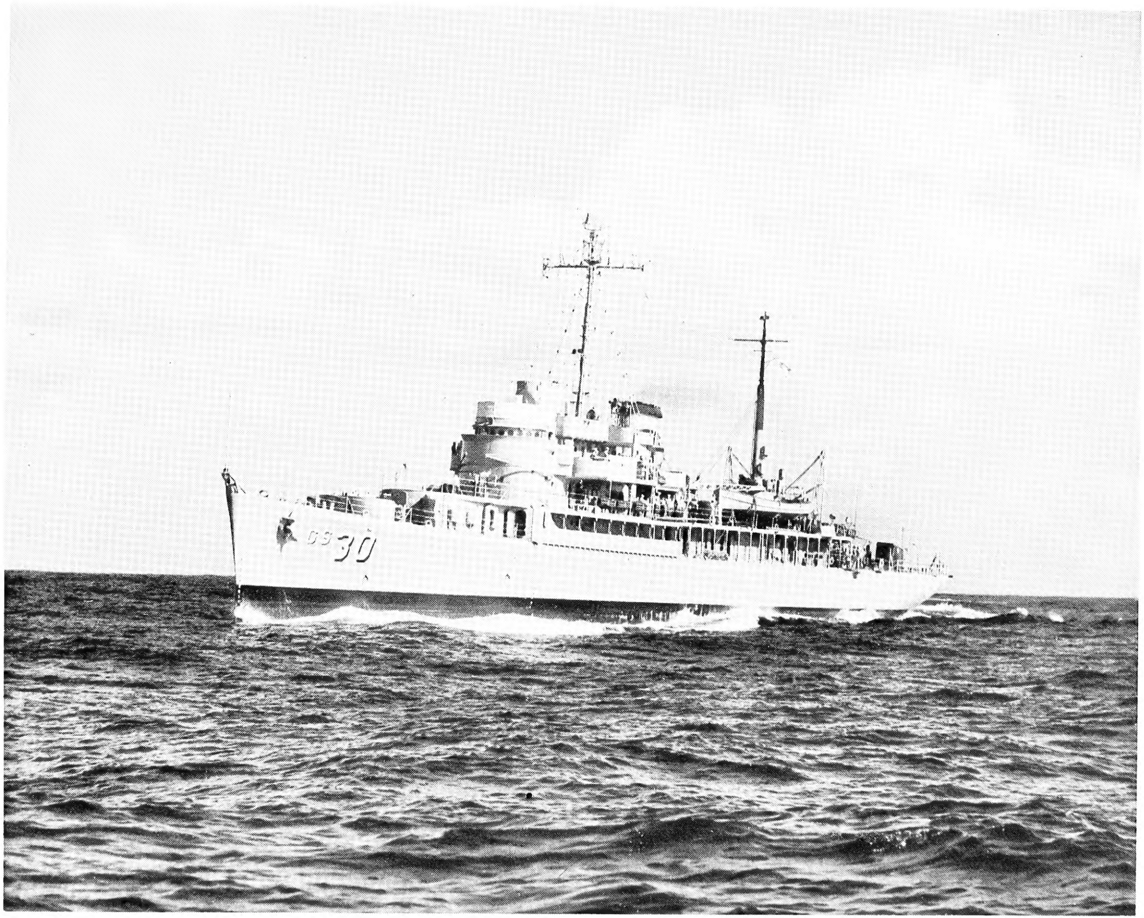
ABSTRACT

Shear strength, expressed as cohesion, was measured by compression and laboratory vane tests on short, usually continuous samples of 30 sediment cores collected by the Hydrographic Office from water depths of 400 to 5,120 m in 8 areas of the North Atlantic, West Mediterranean, and Central Pacific. Clayey silt- and silty clay-size sediments were predominantly of terrigenous origin. The least cohesion measured is about 4.2g/cm^2 and the maximum 234g/cm^2. Although cohesion usually increased with depth in the cores, fluctuations in the strength-depth profile are the rule rather than the exception. Validity of strength data, in light of disturbance caused by both piston and gravity core sampling and other forms of disturbance, was considered with the conclusion that the cohesions reported are sufficiently reliable for engineering use at the present time. Shear strength and laboratory-determined consolidation data are applied, with numerical examples, to the computation of sea-floor sediment ultimate bearing capacity and consolidation under structural loads.



FRONTISPIECE. THE USS SAN PABLO, A HYDROGRAPHIC OFFICE OCEANOGRAPHIC SURVEY SHIP. (U.S. Navy Official Photo.)

FOREWORD

Oceanographers and engineers are becoming increasingly inquisitive about the mass physical properties of deep-sea sediments for academic and practical reasons. Although a large amount of information exists on the mass physical and engineering properties of terrestrial and shallow water marine sediments, very little is known about these properties for sediments of the continental slope and deep-sea floor.

In 1958 an exploratory program to investigate soil mechanics properties in a number of oceanic areas was initiated by the Hydrographic Office under the direction of Dr. Richards. Results of this study are presented in this report through the cooperation of three establishments within the Department of the Navy, the Hydrographic Office, Bureau of Yards and Docks, and Navy Electronics Laboratory.

E. C. Stephan

E. C. STEPHAN
Rear Admiral, U. S. Navy
Hydrographer

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I. INTRODUCTION

This paper is the first of three Hydrographic Office reports describing results of an investigation of engineering and mass physical properties in 35 sediment cores collected from Hydrographic Office oceanographic ships (Frontispiece) in 1958 and 1959. Principal objectives of this paper are to acquaint oceanographers, engineers, and military personnel unfamiliar with soil mechanics[1] with: (1) methods of core collection and testing for physical analysis of sediments, (2) the range of measured shear strength in 30 cores, strength versus depth relations, and general validity of the measurements in light of factors affecting strength of sediments, (3) the practical application of using shear-strength information to compute ultimate bearing capacity of sea-floor sediments, and (4) the practical aspects of consolidation of sea-floor sediments under applied loads.

The second report in the series (Richards, in preparation) describes and discusses the mass physical properties of the 35 cores investigated in this study and relates these properties to depth. Tabulated laboratory test and computed data are contained in the second report. A third report considers computed and laboratory measured consolidation of these sediments.

The first series of cores collected expressly for soil mechanics and soil physics tests were obtained in the spring of 1958. In June, arrangements were made by the Hydrographic Office to have cores tested in the Soil Mechanics Laboratory of the Navy's Bureau of Yards and Docks (BUDOCKS), and testing began the same month. Since then, many more cores were added to the test program at BUDOCKS. Analysis of results of laboratory tests began during 1958. This paper was started during the same period while I was an oceanographer with the Hydrographic Office; it has been concluded during a leave-of-absence while I was a National Academy of Sciences-National Research Council Postdoctoral Resident Research Associate* at the U. S. Navy Electronics Laboratory in San Diego, California[2].

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[1]Skempton (1953) and Terzaghi (1955) are recommended for a geological orientation to some fundamentals of soil mechanics; those readers wishing a more exhaustive treatment are referred to standard texts: for example, Capper and Cassie (1960), Hough (1957, Spangler (1960), Taylor (1948), Terzaghi (1943), Terzaghi and Peck (1948), and Tschebotarioff (1951).

[2]At NEL a decade ago the problem of bearing capacity of fine-grained sediments was considered briefly (Butcher, 1951).

*The author was assigned recently to the Office of Naval Research Branch Office, London W.1, England.

Soil mechanics terms and symbols used herein, in general, conform to those published in 1958 by the joint Committee on Glossary of Terms and Definitions in Soil Mechanics of the American Society of Civil Engineers (ASCE) and the American Society for Testing Materials (ASTM). Definitions in accordance with this glossary are referenced by the letters ASCE in parentheses. The word "sediment" is used in synonymy with the word "soil." Notation is given in Table 1.

It is my pleasure to acknowledge the cooperation of BUDOCKS personnel in this investigation; indeed, it would have been extremely difficult without their assistance. Mr. L. A. Palmer, Soil Mechanics and Paving Consultant (now retired) and Mr. P. P. Brown of BUDOCKS made laboratory facilities available to the Hydrographic Office and generously contributed in many ways to this study. All tests and measurements given in this report, unless otherwise noted, were made in the BUDOCKS Soil Mechanics Laboratory under the capable direction of Mr. C. M. Yeomans. I am indebted to Mr. Yeomans for his continued interest in this work, many excellent suggestions on how to improve different phases of the study, and for numerous knowledgeable discussions that constituted a practical education in soil mechanics.

Helpful comments and suggestions to a preliminary copy of the manuscript were made by Dr. R. J. Hurley, Bell Telephone Laboratories, Whippany, N. J., Messers. Brown and Yeomans of BUDOCKS, Mr. G. H. Keller of the Hydrographic Office, and Mr. D. G. Moore and Dr. E. L. Hamilton, Navy Electronics Laboratory. I appreciate the opportunity afforded by the Navy Electronics Laboratory to study and report these data, and for considerable drafting and typing services. I, of course, assume responsibility for all conclusions resulting from this study.

TABLE 1. NOTATION

A	Area
b	half width of a strip load
C_a	area ratio
C_i	inside clearance ratio
C_o	outside clearance ratio
c	cohesion
c_v	coefficient of consolidation
D_e	minimum inside diameter of core barrel or core liner
D_s	minimum inside diameter of core nose
D_t	maximum outside diameter of core barrel
D_w	maximum outside diameter of core nose
d	depth of load below surface or, depth
e	void ratio
e_f	final average void ratio
e_i	initial average void ratio
F	spring factor
H	corer penetration distance or, thickness of compressible stratum
H_i	initial thickness of compressible stratum
h	vane height or, sample height in compression test
Δh	change of sample height
L_g	core gross length
m & n	coefficients for finding p_z
N	factor corresponding to percentage completion of consolidation
N_c, N_q, N_γ	bearing capacity factors
p	pressure
p_c	compressive strength
p_z	pressure in direction of the vertical axis

TABLE 1. NOTATION (Cont'd)

q_o	ultimate bearing capacity
R_g	gross core recovery ratio
r	vane radius
S	settlement
S_t	sensitivity
s	shear strength
t	time
ΔT	difference between initial and final torque readings in vane test
u	neutral stress
V_s	volume of solid particles
V_v	volume of voids
x & y	width and length of footing
ρ	density (wet unit weight) of sediment
σ	total stress
$\bar{\sigma}$	effective stress
ϕ	angle of shearing resistance

II. SAMPLE COLLECTION AND TEST METHODS

A. CORE COLLECTION

Thirty-five sediment cores were collected from eight different areas of the continental shelf, continental slope, and deep-sea floor in the North Atlantic Ocean, West Mediterranean Sea, and central Pacific Ocean (Fig. 1). Relation of cores within each area is shown in Figure 2. Five cores from Area G had too few strength tests to warrant inclusion in this report; mass physical properties of these, and the other 30 cores, are given elsewhere (Richards, in preparation).

Table 2 summarizes pertinent information about each core and the corer it was obtained with. In this table, names "Kullenberg corer" (Kullenberg, 1947), "Phleger corer" (Phleger and Parker, 1951, p. 3-5), and "Ewing corer" (Heezen, 1952) refer to those types of corers used and described by the Hydrographic Office (1955, p. 54-66). The Hydroplastic corer (Fig. 3), utilizing a barrel made of high-impact grade polyvinyl chloride (PVC) plastic without an inner liner, was designed at the Hydrographic Office for use in this program (Richards, 1960; Richards and Keller, in press). All corers routinely were used with spring-leaf core catchers.

Each core studied is composed predominantly of one sediment type and is classified in Table 2 by means of a mechanical analysis and use of the triangular diagram (Fig. 4) system of nomenclature developed by Shepard (1954). The analytical procedure is described in the second report.

Kullenberg and Phleger corers used by the Hydrographic Office have a cellulose acetate butyrate liner for sediment retention that fits inside a metal core barrel. The Ewing corer and the Hydroplastic corer do not have liners, and sediment is retained in the metal barrel of the former and the PVC plastic barrel of the latter. Loss of sediment interstitial water from metal is nil and from PVC negligible. Water loss through cellulose acetate butyrate can be appreciable (Keller and others, in press). A means of protecting these cores from desiccation was provided while they were stored on the ship and transported to the laboratory. In the beginning part of the program, core liners were packed in wet rags. Later, core liners were coated in a specially constructed wax bath, that allowed the core to remain upright during the coating process (Keller and others, in press), using microcrystaline wax. This wax is somewhat less pervious to water than paraffin and does not readily chip or crack. Most cores were protected against mechanical vibration and shock and were shipped to the laboratory in an upright position.

Cores longer than 120 to 150 cm (4 to 5 feet) were sectioned by careful sawing prior to shipment. Piston cores D 1 and H 12 were returned to the laboratory in a

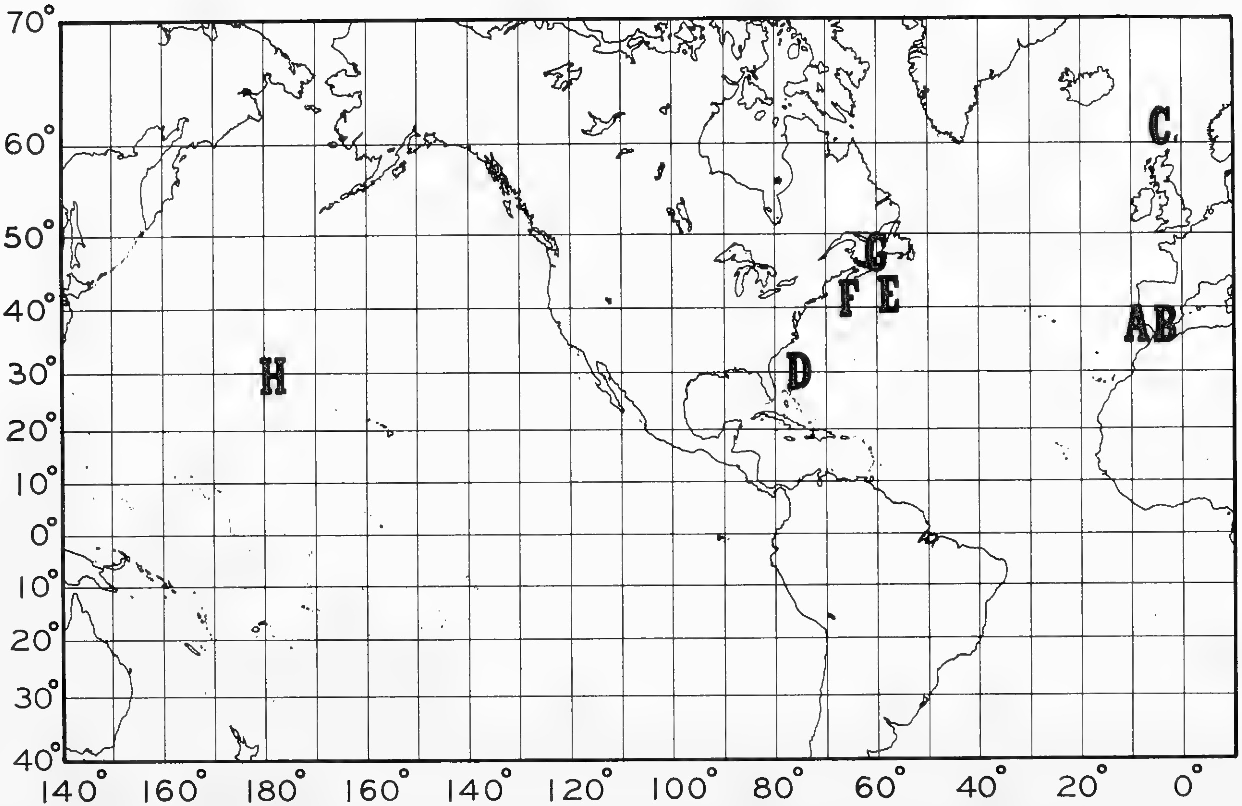


FIGURE 1. LOCATION OF AREAS FROM WHICH CORES WERE COLLECTED

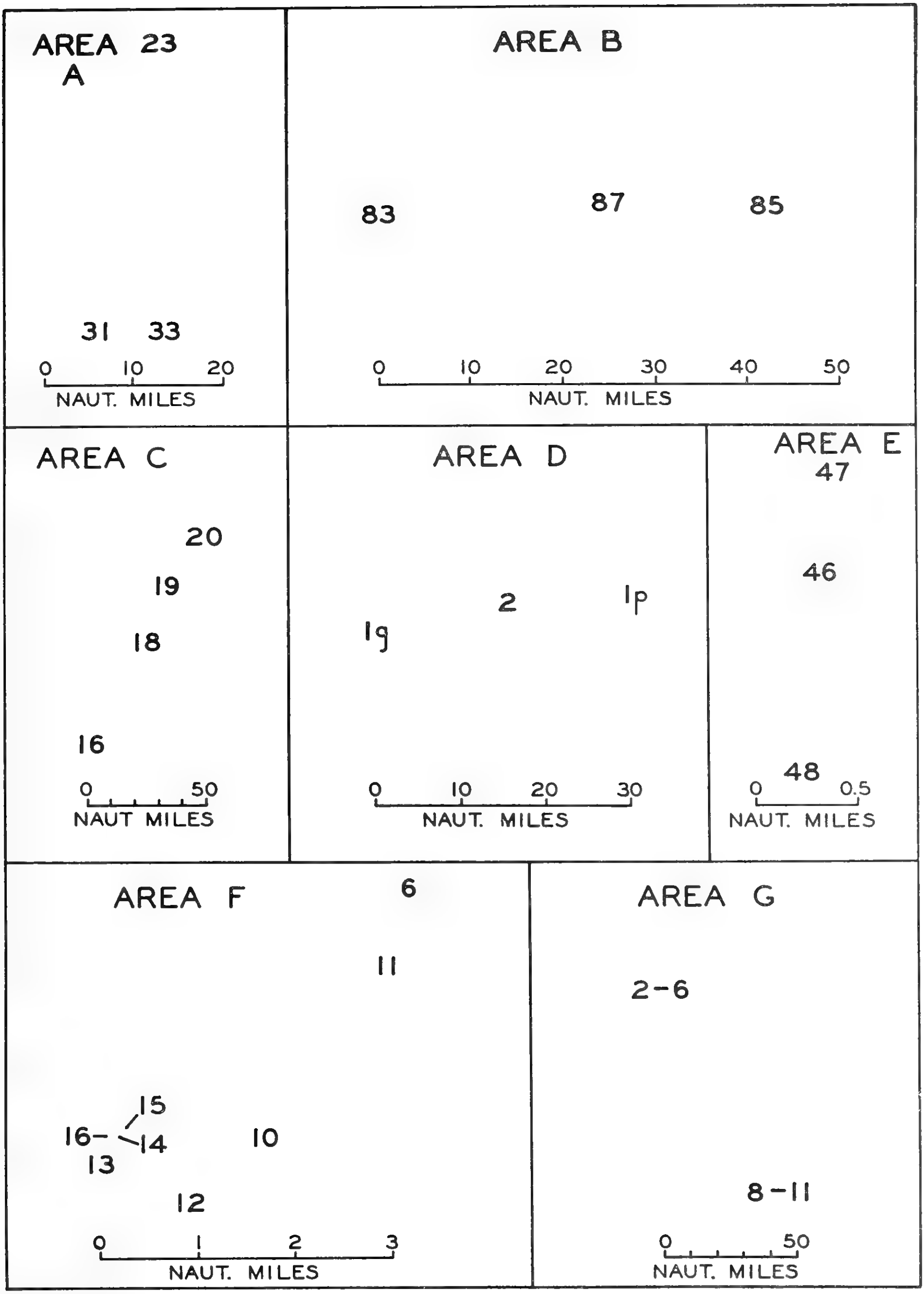


FIGURE 2. LOCATION OF CORES WITHIN EACH AREA

TABLE 2. CORE AND CORER SUMMARY

Area	Core (No.)	Approx. Water Depth[1] (m)	Predominant Sediment Type[2]	Core Diameter (cm)	Core Length (cm)	Estimated Penetration (cm)	Gross Recovery Ratio[3] (%)	Corer Type[3] G = Gravity P = Piston	Inside Clearance Ratio (%)	Outside Clearance Ratio (%)	Area Ratio (%)	Collector
A	23	1125	T, clayey silt	4.75	76	150	50	Kullenberg-G	3.6	10.1	105.5	A.F. Richards
	31	400	T, clayey silt	4.75	104	150	68	Kullenberg-G	3.6	10.1	105.5	"
	33	1060	T, silty clay	4.75	97	150	63	Kullenberg-G	3.6	10.1	105.5	"
B	83	1315	T, silty clay & clayey silt	4.75	102	180	57	Kullenberg-G	3.6	10.1	105.5	A.F. Richards
	85	730	T, clayey silt	4.75	79	180	43	Kullenberg-G	3.6	10.1	105.5	"
	87	1125	T, clayey silt	4.75	81	180	44	Kullenberg-G	3.6	10.1	105.5	"
C	16	1115	T, clayey silt	4.75	79	180	100	Kullenberg-P	3.6	10.1	105.5	A.F. Richards
	18	1115	T, clayey silt	4.75	117	170	100	Kullenberg-P	3.6	10.1	105.5	"
	19	1460	T, clayey silt	4.75	76	120	100	Kullenberg-P	3.6	10.1	105.5	"
	20	1610	T, clayey silt	4.75	51	120	100	Kullenberg-P	3.6	10.1	105.5	"
D	1	1240	C&T, clayey silt	2.5	36	75	60	Phleger-G	10.2	0	62.2	S.W. Oliver
	2	2010	C&T, sand-silt-clay	2.5	30	90	33	Phleger-G	10.2	0	62.2	"
	1	2560	C, clayey silt	6.35	511	610	100	Ewing-P	0.6	22.7	84.5	A.F. Richards
E	46	2010	T, clayey silt	4.75	142	180	78	Kullenberg-G	3.6	10.1	105.5	G.H. Knoop, Jr.
	47	2010	T, clayey silt	4.75	157	180	86	Kullenberg-G	3.6	10.1	105.5	"
	48	2195	T, clayey silt	4.75	109	180	60	Kullenberg-G	3.6	10.1	105.5	"

[1] Uncorrected sonic depth. One meter equals 0.547 fathoms or 3.28 U.S. feet.

[2] T = terrigenous source, C = calcareous source, and PC = pelagic clay source of material.

[3] See text.

TABLE 2. CORE AND CORER SUMMARY (Cont'd)

Area	Core (No.)	Approx. Water Depth[1] (m)	Predominant Sediment Type[2]	Core Diameter (cm)	Core Length (cm)	Estimated Penetration (cm)	Gross Recovery Ratio[3] (%)	Corer Type[3] G = Gravity P = Piston	Inside Clearance Ratio (%)	Outside Clearance Ratio (%)	Area Ratio (%)	Collector
F	6	2270	T, clayey silt	8.2	269	280	100	Hydroplastic-G	1.3	13.4	56.3	A.F. Richards
	10	2450	T, clayey silt	4.75	175	180	100	Kullenberg-G	3.6	10.1	105.5	"
	11	2450	T, clayey silt	8.2	152	-	-	Hydroplastic-G	1.3	13.4	56.3	"
	12	2430	T, clayey silt	4.75	168	170	96	Kullenberg-G	3.6	10.1	105.5	"
	13	2415	T, clayey silt	4.75	142	140	100	Kullenberg-G	3.6	10.1	105.5	"
	14	2395	T, silty clay	4.75	173	140	100	Kullenberg-G	3.6	10.1	105.5	"
	15	2415	T, silty clay	4.75	155	120	100	Kullenberg-G	3.6	10.1	105.5	"
	16	2415	T, silty clay	4.75	170	130	100	Kullenberg-G	3.6	10.1	105.5	"
G	2	455	T, silty clay	6.35	557	610	100	Ewing-P	0.6	22.7	84.5	G.H. Keller
	6	455	T, silty clay	8.1	152	210	71	Hydroplastic-G	1.3	13.4	56.3	"
	8	455	T, silty clay & clayey silt	8.1	244	300	100	Hydroplastic-P	1.3	13.4	56.3	"
	10	455	T, silty clay	8.1	81	240	75	Hydroplastic-G	1.3	13.4	56.3	"
H	12	5120	PC, silty clay	6.35	511	-	100	Ewing-P	0.6	22.7	84.5	R.H. Michel
	13	5120	PC, silty clay	4.75	142	-	-	Kullenberg-G	3.6	10.1	105.5	"

FIGURE 3. ORIGINAL MODEL OF HYDROPLASTIC CORER RIGGED FOR GRAVITY-TYPE OPERATION

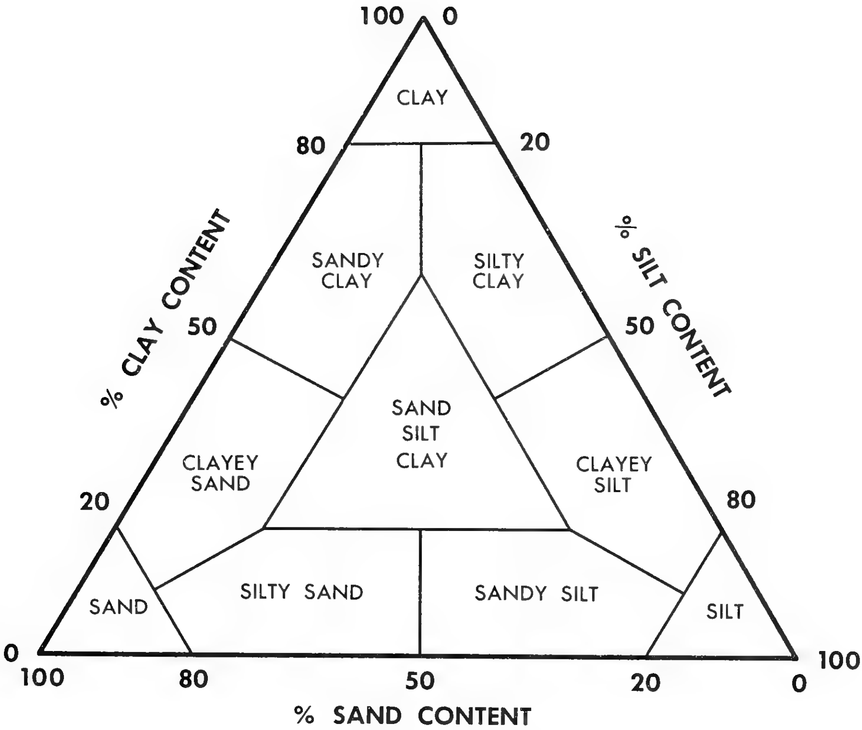


FIGURE 4. NOMENCLATURE OF SEDIMENT TYPES. (After Shepard, 1954, p. 157)

horizontal position; they were not packed to reduce the effects of vibration and shock.

Until tested all cores except those in metal barrels were stored upright at the BUDOCKS soils laboratory in a room of about 98 percent relative humidity.

B. SAMPLING PROBLEMS

General statement -- Two fundamental questions confront those who measure shear strength in submarine sediment cores: (1) how is the thickness and depth of any given stratum in a core related to its in-place thickness and depth, and (2) how much disturbance to the sedimentary structure is caused by the sampling operation? Unfortunately, neither question is easily answered.

The gross recovery ratio (Hvorslev, 1949, p. 100), R_g, defines the gross length of core recovered, L_g, in terms of the depth penetrated by the corer, H.

$$R_g = \frac{L_g}{H} \tag{1}$$

A nearly sacrosanct assumption made by many marine geologists is that piston samplers usually collect undisturbed samples with gross recovery ratios of 100 percent (see: Kullenberg, 1947, p. 14; Ericson and Wollin, 1956, p. 107; Ewing, 1960, p. 191; and Ericson and others, 1961, p. 196). Another common assumption is that core shortening inevitably occurs in gravity cores and the gross recovery ratio is less than 100 percent. Recent investigations suggest that both assumptions are in need of revision.

Principles and mechanics of gravity and piston coring are considered in detail elsewhere (Piggot, 1941; Hvorslev and Stetson, 1946, p. 938-943; Kullenberg, 1947, p. 23-32, and 1955, p. 53-76; and Hvorslev, 1949, p. 93-110). Hence, our attention immediately can be directed to the two principal problems stated above, following the presentation of a few helpful ratios defined by Hvorslev (1949, p. 105-109) that effect corer performance.

The inside clearance ratio, C_i, controls inside friction

$$C_i = \frac{D_s - D_e}{D_e} \tag{2}$$

where D_s is the minimum inside diameter of the core barrel or core liner and D_e is the minimum inside diameter of the core nose (core cutter). Outside friction is controlled

by the outside clearance ratio, C_o,

$$C_o = \frac{D_w - D_t}{D_t} \tag{3}$$

where D_w is the maximum outside diameter of the core nose and D_t is the outside diameter of the core barrel. The ratio of the volume of displaced sediment to the volume of the sample is the area or Kerf ratio, C_a.

$$C_a = \frac{D_w^2 - D_e^2}{D_e^2} \tag{4}$$

Core shortening -- Three different interpretations are shown graphically in Figure 5 to relate the location of an increment of cored sample to its in-place location: (I) The length of core recovered is equal to the same distance below the water-sediment interface irrespective of how far the corer penetrated the bottom, an increment of core represents the same increment in place, and the gross recovery ratio is 100 percent. This interpretation has been assumed for piston submarine cores, as previously mentioned. (II) A linear relationship exists between the increment of core recovered and the distance penetrated, each increment of core represents a less-thick increment in place, and the gross recovery ratio is always less than 100 percent. This interpretation has been demonstrated by Emery and Dietz (1941, p. 1706-1711) and usually is favored for gravity corers (see, for example: Ericson and Wollin, 1956, p. 107). (III) Little or no core shortening occurs down to a certain distance, about 40 to 75 cm (15 to 30 in) according to data presented by Hvorslev (1949, p. 105-119) - over which distance the gross recovery ratio is about 100 percent; below this distance the cumulative shortening is proportional to the additional distance penetrated, and the gross recovery ratio is always less than 100 percent. This interpretation is favored by Hamilton (1960, p. 377) for all gravity cores. A fourth interpretation is a modification of the third: increments of core become successively smaller from top to bottom compared to their thickness in place (Pratje, 1934; Piggot, 1941); a plot would fall between II and III in Figure 5.

Kallstenius (1958, p. 9-10) reports that soft strata between rigid strata may be squeezed out in piston samples without it being possible to detect the phenomenon; his tests showed that 20 percent deformation (core shortening) in a compression test produced distortion detectable with difficulty. The assumption that piston submarine core recovery ratios are 100 percent, in consequence, is questionable.

The amount of core shortening in gravity cores appears to be a function of corer design, the better the engineering design the longer the core with 100 percent gross recovery ratio. Richards and Keller (in press) have shown that in a large diameter

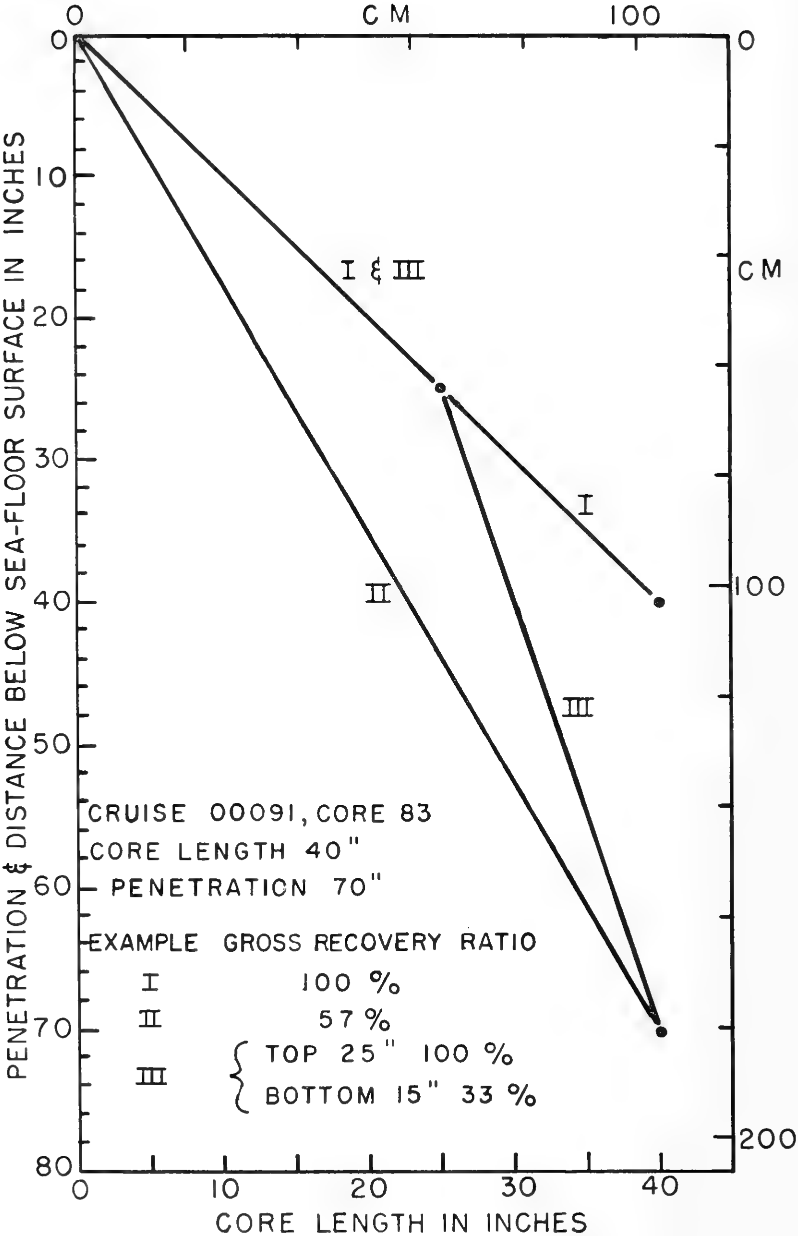


FIGURE 5. GRAPH SHOWING THREE WAYS TO RELATE CORE LENGTH TO CORER PENETRATION

Hydroplastic gravity core no evidence for shortening occurred in the top approximately 50 cm. It may be possible to take longer gravity cores with a 100 percent gross recovery ratio using even better engineering design.

Sampling disturbance -- Results of tests on varied clays led Hvorslev (1949, p. 105-109) to recommend that long samplers in cohesive sediments should have an inside clearance ratio of 0.75 to 1.5 percent, an outside clearance ratio of less than 3 percent, and an area ratio less than 10 percent; a greater area ratio was considered permissible if the sampler had a stationary piston and/or a very small cutting edge angle.

It was not possible to easily test Hvorslev's recommendations on dimensions of corers until the past decade, when in-place strength tests made by the vane borer (bibliography in Osterberg, 1957) were compared to different types of samplers. Two important papers by Jakobson (1954) and Kallsteius (1958) resulted from extensive tests conducted on relatively homogeneous post-glacial clays by the Royal Swedish Geotechnical Institue. These tests in general confirmed Hvorslev's recommendations and showed that piston samplers obtained undisturbed samples only if properly engineered. Kallstenius (1958, p. 67) emphasized the importance of a small cutting-edge angle. Most corers used by oceanographers have large edge angles. In Norway, Bjerrum (1954a, p. 55) reported that a redesigned thin-wall piston sampler having a small area ratio successfully obtained undisturbed samples in sensitive clays.

Photographs or drawings of strata in piston cores (Arrhenius, 1952, appendix; Ericson and Wollin, 1956, Fig. 4; Sykes, 1960, Figs. 28-30; and others) amply show that submarine piston cores are not undisturbed; inspection of clearance and area ratios (Table 3) clearly infers why.

Small diameter, poorly engineered (for undisturbed sampling) gravity cores probably show disturbance resulting from core shortening of Type II in Figure 5. It is noteworthy that better engineered, larger-diameter gravity corers do not show core shortening near the surface (Richards and Keller, in press), and hence take more nearly undisturbed samples. The Hvorslev-Stetson (1946) gravity corer also may take little-disturbed samples because of its good design (Table 3).

A detailed analysis of disturbance caused by sampler shape has been written by Kallstenius (1958, p. 40-66), should the reader desire additional information on how disturbance is caused.

Summary -- It has not been demonstrated that piston cores taken by oceanographers have gross recovery ratios of 100 percent. Tests and experience reported by Scandinavian soil engineers suggests that sample disturbance, caused by excessive clearance and area ratios and large taper angle of the cutting edge of these corers, could be reduced by applying modern engineering techniques.

TABLE 3. CLEARANCE AND AREA RATIOS OF SOME SUBMARINE SEDIMENT CORERS[1]

Corer	Piston or Gravity Type	Clearance Ratios Inside (%)	Clearance Ratios Outside (%)	Area Ratio (%)	Reference
Hvorslev-Stetson	Gravity	1, 1.6, 2	1	35	Hvorslev & Stetson (1946)
USNHO Hydroplastic	Either	1.3	13	57	Richards & Keller (in press)
USNHO "Phleger"	Gravity	10.2	0	62	U.S. Hydro. Off. (1955)
USNHO "Ewing"	Piston	0.6	23	84	U.S. Hydro. Off. (1955)
Lamont Geol. Obs. Ewing	Piston	5.3	18	87	Heezen (1952)
USNHO "Kullenberg"	Either	1.8	10	105	U.S. Hydro. Off. (1955)
USNEL Standard	Either	2.4	22	130	Richards & Keller (in press)
Kullenberg	Piston	0	25	373	Kullenberg (1955)
Hvorslev's recommendations: cohesive sediments and long corers.		0.75 to 1.5	<3	<10	Hvorslev (1949)

Most gravity cores taken by oceanographers appear to have gross recovery ratios much less than 100 percent, with consequent sample disturbance; the magnitude of the ratio appears to be a function of the corer design. Gravity cores with gross recovery ratios of 100 percent, at least in the upper 50 cm or so, can be obtained by better design.

In this report, the distance below the top of the core is the actual distance measured in the laboratory from the surface of each core. The gross recovery ratio listed in Table 2 is based on the extreme condition that core shortening is the same throughout the length of the core (Fig. 5, II), and that core penetration equals corer penetration. Corer penetration, incidentally, was crudely estimated by the conventional method of measuring the amount of sediment adhering to the outside of the core barrel after the corer had been brought aboard the ship. The other extreme is that the gross recovery ratio is 100 percent, which has been assumed for all piston cores listed in Table 2.

[1]After Richards and Keller (in press), with additional computations by Richards.

C. LABORATORY PROCEDURE

General Statement -- Briefly summarized, the laboratory test program was to: (1) carefully cut the plastic liner, or barrels of metal or plastic, into lengths of about 15 cm (6 in), (2) push 5.1 cm (2 in) long, thin-wall brass tubes of 4.9 cm (1.94 in) diameter into each length for measurement of mass and volume, (3) gently extrude sediment from each tube for the compression test, and (4) quarter each sample lengthwise following the strength test; the first and third quarters were combined and used to determine water content and the specific gravity of solids, while the second and fourth quarters were combined and used for Atterberg limit tests and mechanical analysis. Vane-shear tests were made in sediment contained in the brass tube. Separate samples were used for consolidation tests. Unless specifically noted later, the 5-cm sample length was standard.

Shear Strength -- A simple compression test device with plastic platens at either end of the axial loading rod (Fig. 6) was used for measurement of unconfined compressive strength. A brief resume of the test steps and ensuing computations used in the BUDOCKS laboratory is given in the appendix. Test data result in a stress-strain relationship (Fig. 7a). Failure of the sample was taken at the point of greatest curvature of the stress-strain line or, if this point was undeterminable, arbitrarily at 20 percent axial strain. Compression strength, p_c, is related to shear strength, s, by

$$s = 0.5\, p_c \qquad (5)$$

A discussion of the theoretical reasons for this relationship is given by Skempton (1942, Appendix 2).

By mid-1959, commercial laboratory vane-shear equipment (Fig. 8) was obtained (from Wykeham Farrance Engineering Ltd.) by the Hydrographic Office and used at BUDOCKS solely, on some cores, or alternatively with the compression test equipment. This machine uses a vane 1.3 cm (0.5 in) in diameter and 1.9 cm (0.75 in) long rotated by a constant-speed motor at a rate of 6 degrees per minute (0.1° per second). The vane was buried in sediment prior to measurement by a distance not less than its length. Test procedure is summarized in the appendix. Results from the test are expressed in a stress-vane rotation relationship (Fig. 7b). Sample failure is determined at the major inflection of the stress-rotation curve. When water-saturated, cohesive sediments are tested at an unaltered water content, the vane test directly measures shear strength. Osterberg (1957) has reviewed the history of the vane test.

Certain samples were completely remolded to destroy all natural structure and again tested to determine the remolding sensitivity (Terzaghi, 1944, p. 613), S_t, at an unaltered water content:

$$S_t = \frac{\text{"undisturbed" strength}}{\text{remolded strength}} \qquad (6)$$

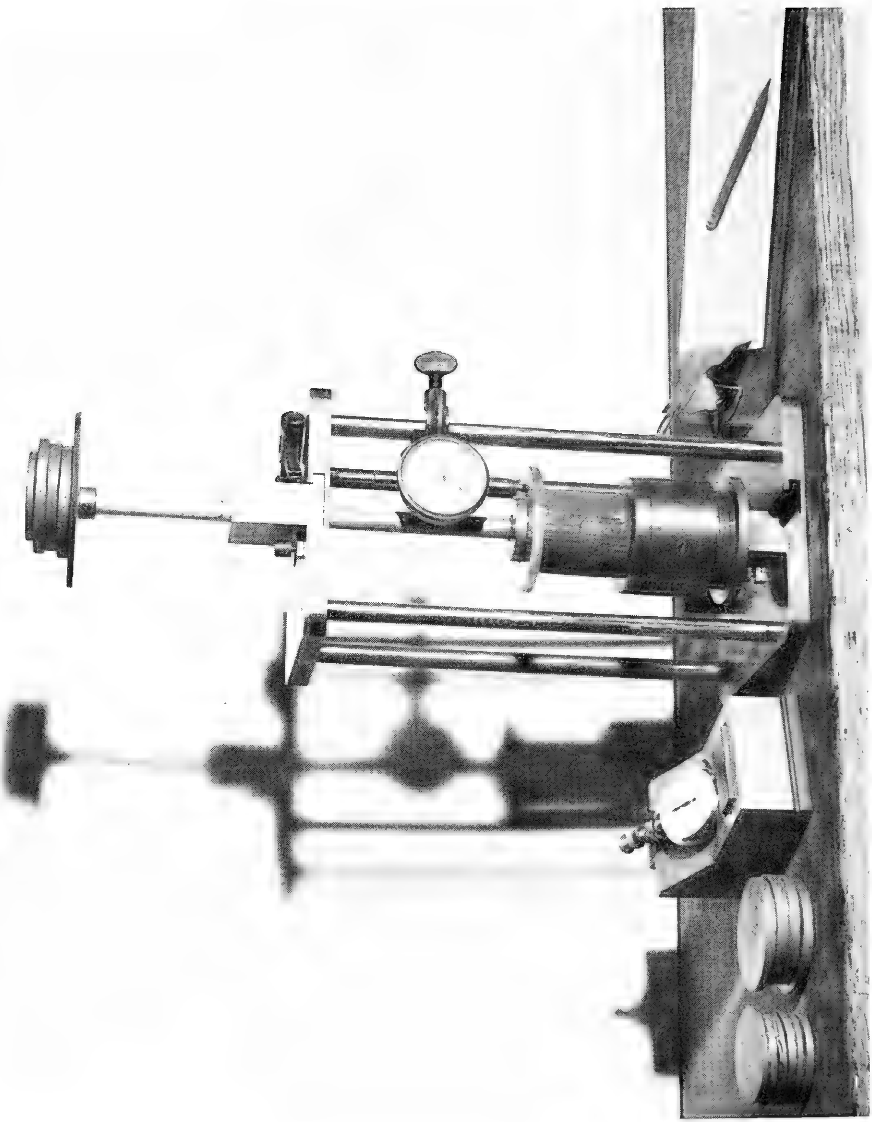


FIGURE 6. COMPRESSION TEST APPARATUS

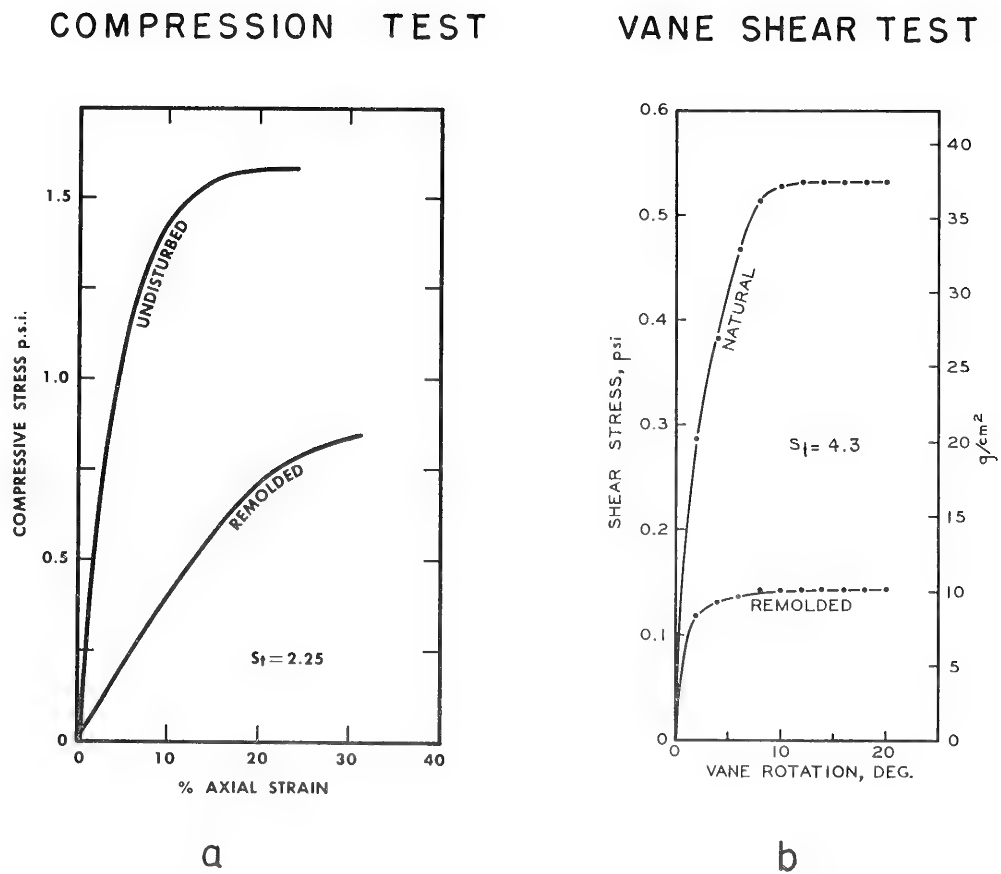


FIGURE 7. RESULTS OF STRENGTH TESTS: (a) STRESS RELATED TO STRAIN FROM A COMPRESSION TEST AND (b) SHEAR STRESS RELATED TO VANE ROTATION IN DEGREES FROM A VANE SHEAR TEST. SENSITIVITY IS DENOTED BY S_t

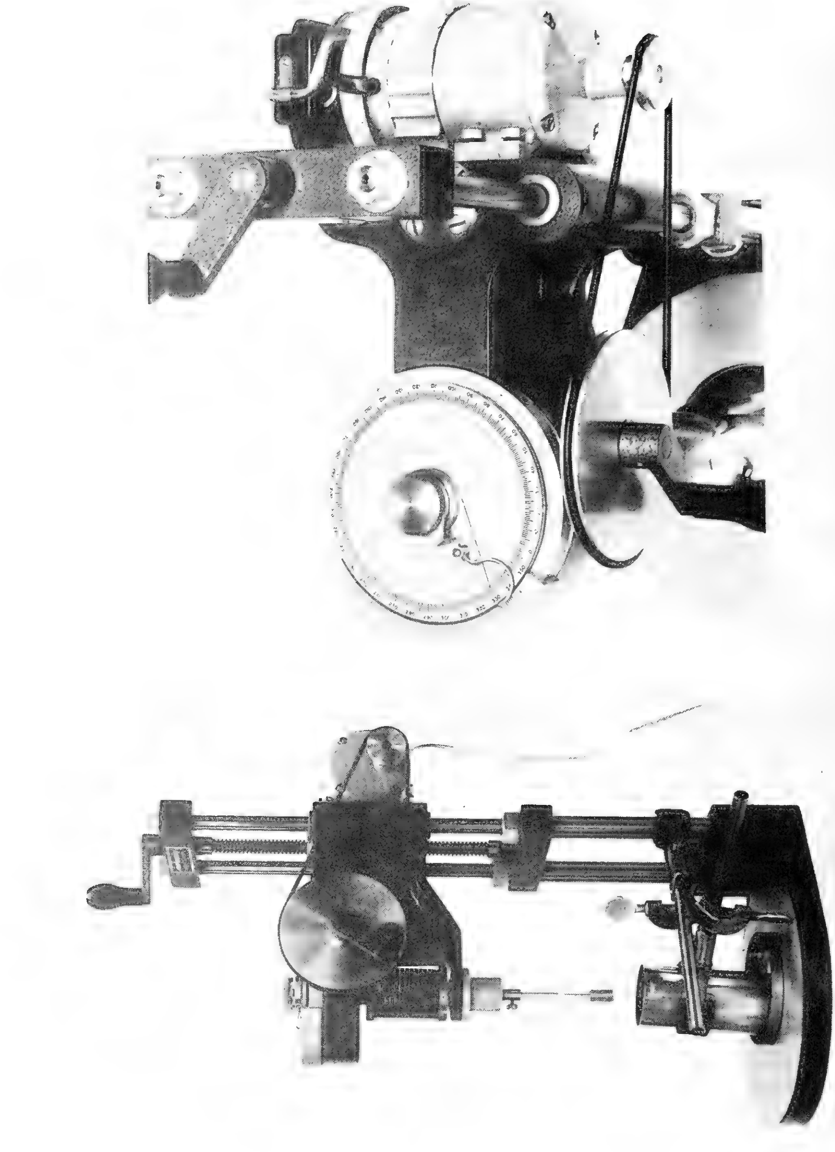


FIGURE 8. VANE SHEAR APPARATUS

In passing, it should be mentioned that it was impractical to use the most popular shear strength test, triaxial shear (Lambe, 1951, p. 122-137; Bishop and Henkel, 1957), on the surficial 3 m or so of the sea-floor sediments examined. It is difficult to obtain a true cylindrical sample of very soft material, and marked variation of strength in short distances precludes using the three contiguous samples that usually are required for valid results.

More information on shear strength measurement can be found in standard texts; a good summary is given by Skempton and Bishop (1950).

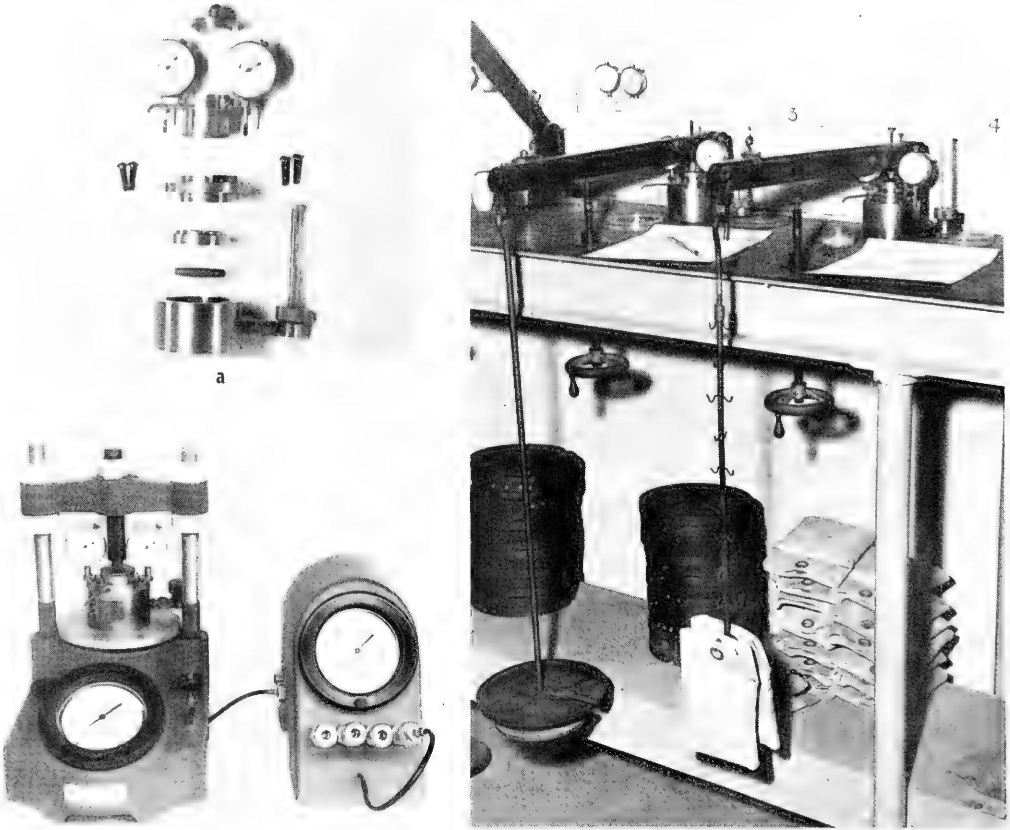
It is appropriate for those who would make strength measurements to remember the warning of Burmister (1946), that every test to determine the strength and other important physical properties of sediments is a research problem; methodology suitable for one set of requirements may not be satisfactory for another.

Consolidation -- The consolidation test is one in which the sample is laterally confined in a ring and compressed between porous plates (ASCE) that allow the escape of interstitial water. At BUDOCKS, the specially designed fixed-ring consolidometer cell accepts a specimen 1.25 cm (0.5 in) thick and 4.9 cm (1.94 in) in diameter (Fig. 9a). Loads are applied either pneumatically (Fig. 9b), or by a lever system on which known weights are hung (Fig. 9c).

The logarithm of time-fitting method was used in the computations of consolidation as a function of time. Test results are presented by plotting the void ratio, e, as a function of the logarithm (to the base 10) of pressure, p, or e-log p curve (Fig. 10). Void ratio is the ratio of the volume of void space, V_v, to the volume of solid particles, V_s, in a given sediment mass (ASCE) or

$$e = \frac{V_v}{V_s} \tag{7}$$

The reader is referred to standard texts (for example, Lambe, 1951, p. 74-87) for more detailed information on consolidation tests. They also will be discussed in the report on consolidation, which is in preparation.



a b c

FIGURE 9. CONSOLIDATION TEST APPARATUS: (a) EXPLODED VIEW OF THE BUDOCKS CONSOLIDOMETER CELL SHOWING EXTENSOMETERS, (b) PNEUMATIC LOADING MACHINE, WITH LOW PRESSURE DIAL ATTACHMENT, AND (c) LEVER SYSTEM LOADING MACHINES

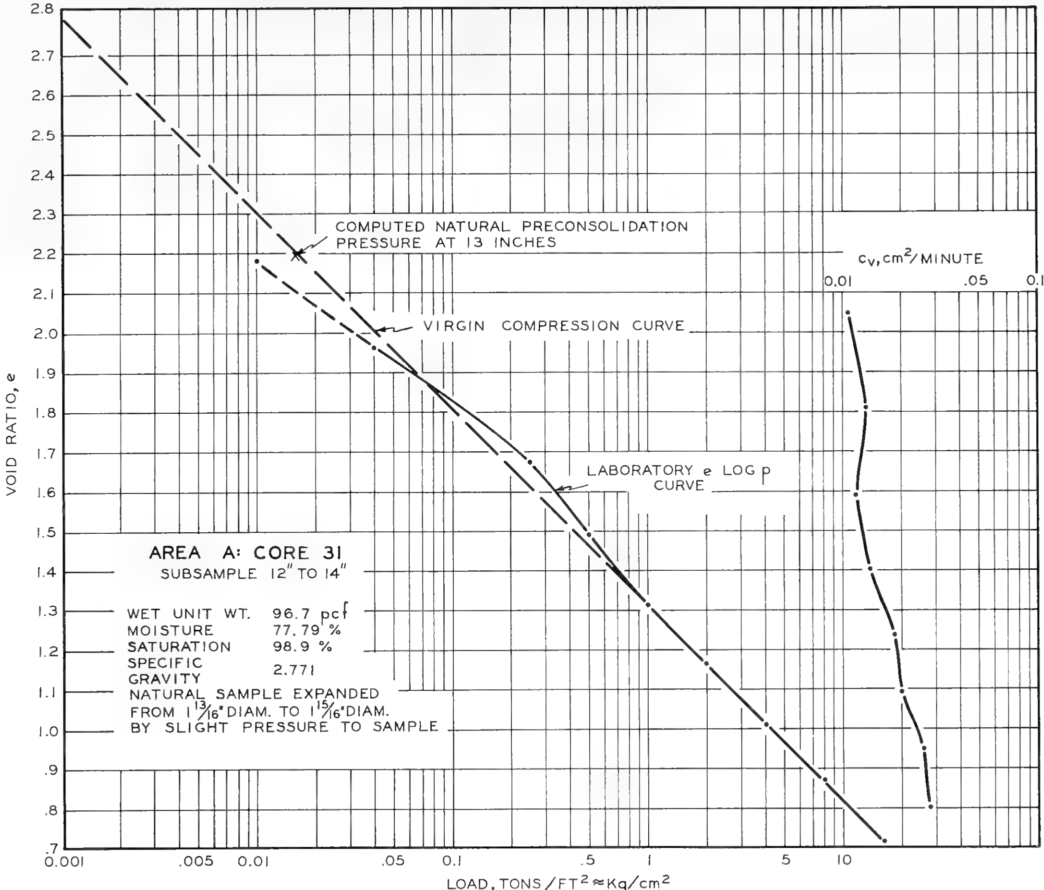


FIGURE 10. VOID RATIO RELATED TO THE LOGARITHM OF PRESSURE FOR CORE A 31

III. COHESION IN CORES

Very little is known about the distribution of shear strength in sediments of the sea floor. While a few studies have been made on sediments in harbors and on the continental shelf, the only investigations made in deeper water appear to be confined to the Pacific Ocean. Arrhenius (1952) made the first study of the distribution of shear strength in a number of cores collected on the Swedish Deep-Sea Expedition using the Swedish State Railways fall-cone pentrometer. Unfortunately, his measurements are expressed as relative strength values and as such are of little use except to show the relationship of strength profiles to depth. (To make these measurements of more universal use, Moore and Richards (submitted) prepared a graph to convert the relative strength values to conventional units in the English and metric systems). More recently, Moore (1959, and in press) published shear strengths of a few selected deep-sea samples in an analysis of bottom-slope stability using results from vane and direct shear tests.

Data are presented (Figs. 11 through 21) relating shear strength, expressed as cohesion, to distance below the sea floor in cohesive sediments that were collected in cores from water depths of 400 to 5,120 m (1,310 to 16,800 feet) in eight areas of the North Atlantic Ocean, West Mediterranean Sea, and Central Pacific Ocean.

Strength measurements were made in pounds per square inch, psi, and converted to g/cm^2. Each compression test value is plotted as if it were a point midway between the ends of the sample. The compression test sample length for all cores is 5 cm, except as follows: Area D, (piston) core 1, 10.5 cm; Area F, core 6, 10.5 cm, core 11, 10 cm; and Area G, all cores, about 10 cm. Values obtained from vane shear tests are representative of only about a 1.9 cm vertical section in any given sample; these values likewise are plotted as if they were taken at a point midway between the ends of the sample.

Vane and compression tests made alternately on succeeding core sections generally show fairly good agreement of values (Fig. 15, cores 47 and 48, and Fig. 18, core 16), although there are exceptions (Fig. 21). In two examples, vane values for an unknown reason are about 15 g/cm^2 less than compression values (Fig. 15, core 46, and Fig. 20). In another instance, both vane and compression values show an apparently erratic distribution (Fig. 21), and it is uncertain which, if either, is correct.

The apparent inconsistency in curve fitting in the different figures is intentional. The degree of fit of a curve to the values shown, insofar as possible, is directly proportional to their accuracy. Deviation of points less than 7 g/cm^2 (0.1 psi) from a curve are insignificant because test precision is about 7 g/cm^2 for compression tests and 0.7 g/cm^2 for vane tests. An example of lack of fit is shown for core D 1g

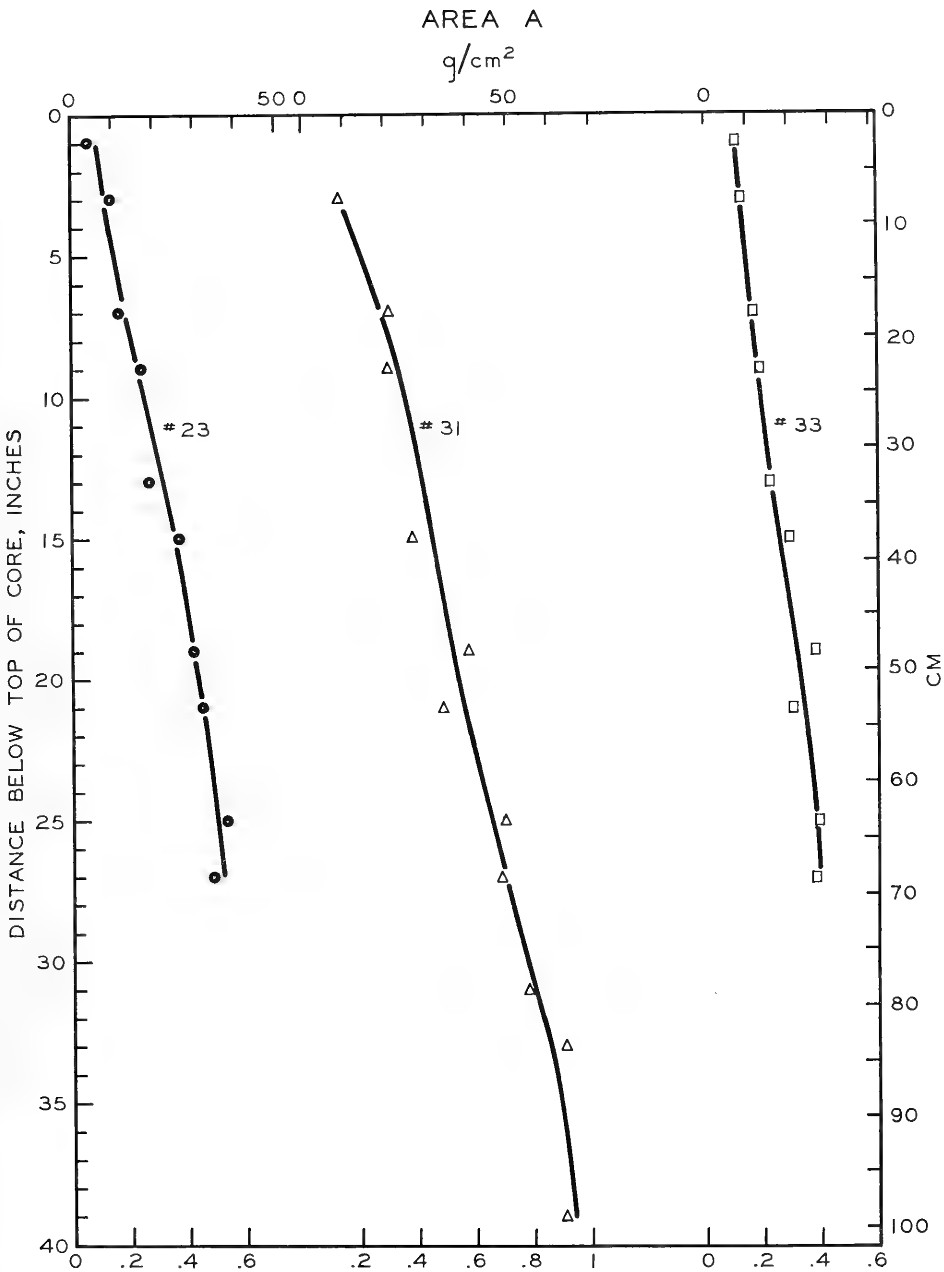


FIGURE 11. DEPTH RELATED TO COHESION (COMPRESSION TESTS): AREA A CORES

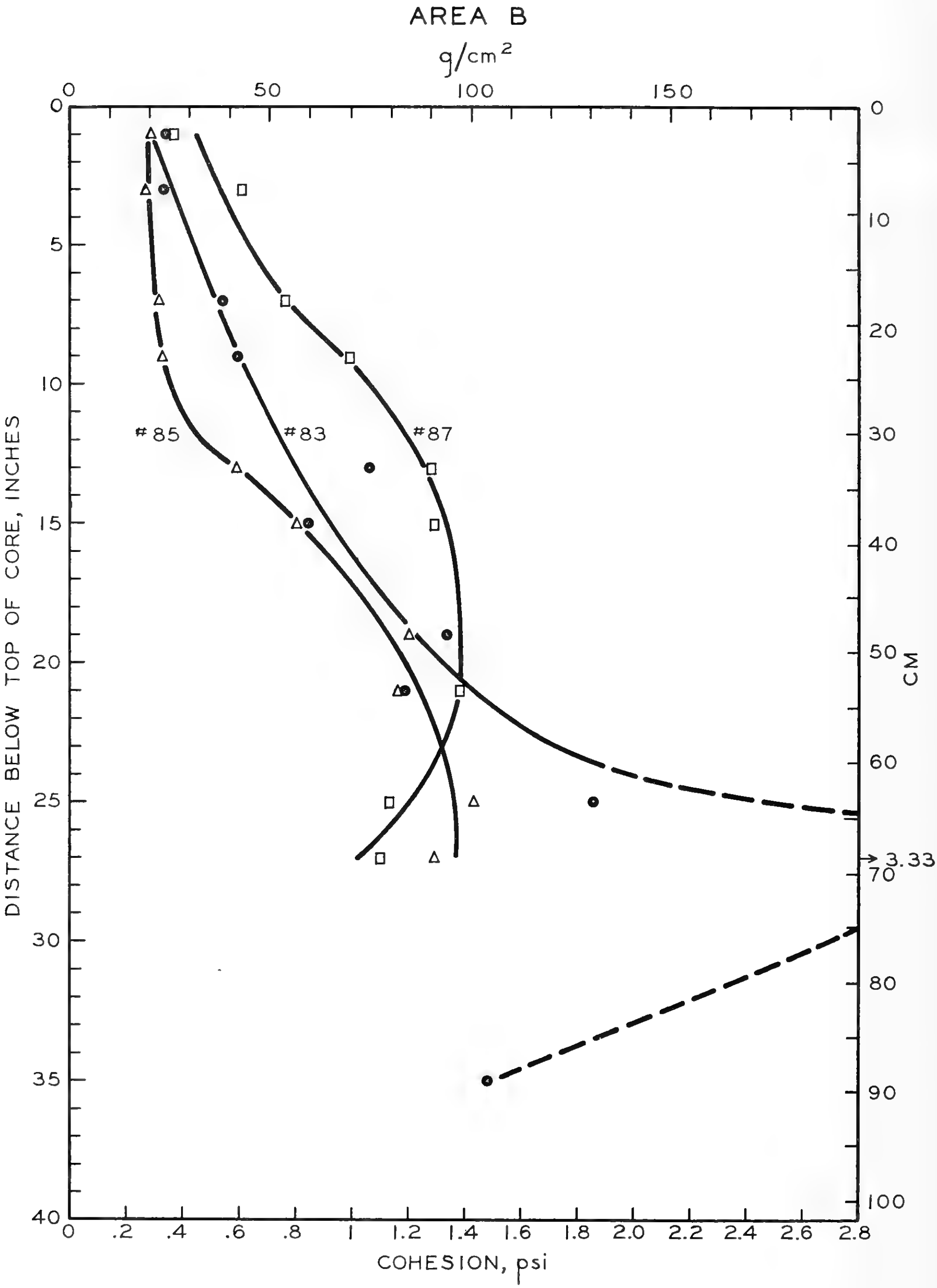


FIGURE 12. DEPTH RELATED TO COHESION (COMPRESSION TESTS): AREA B CORES

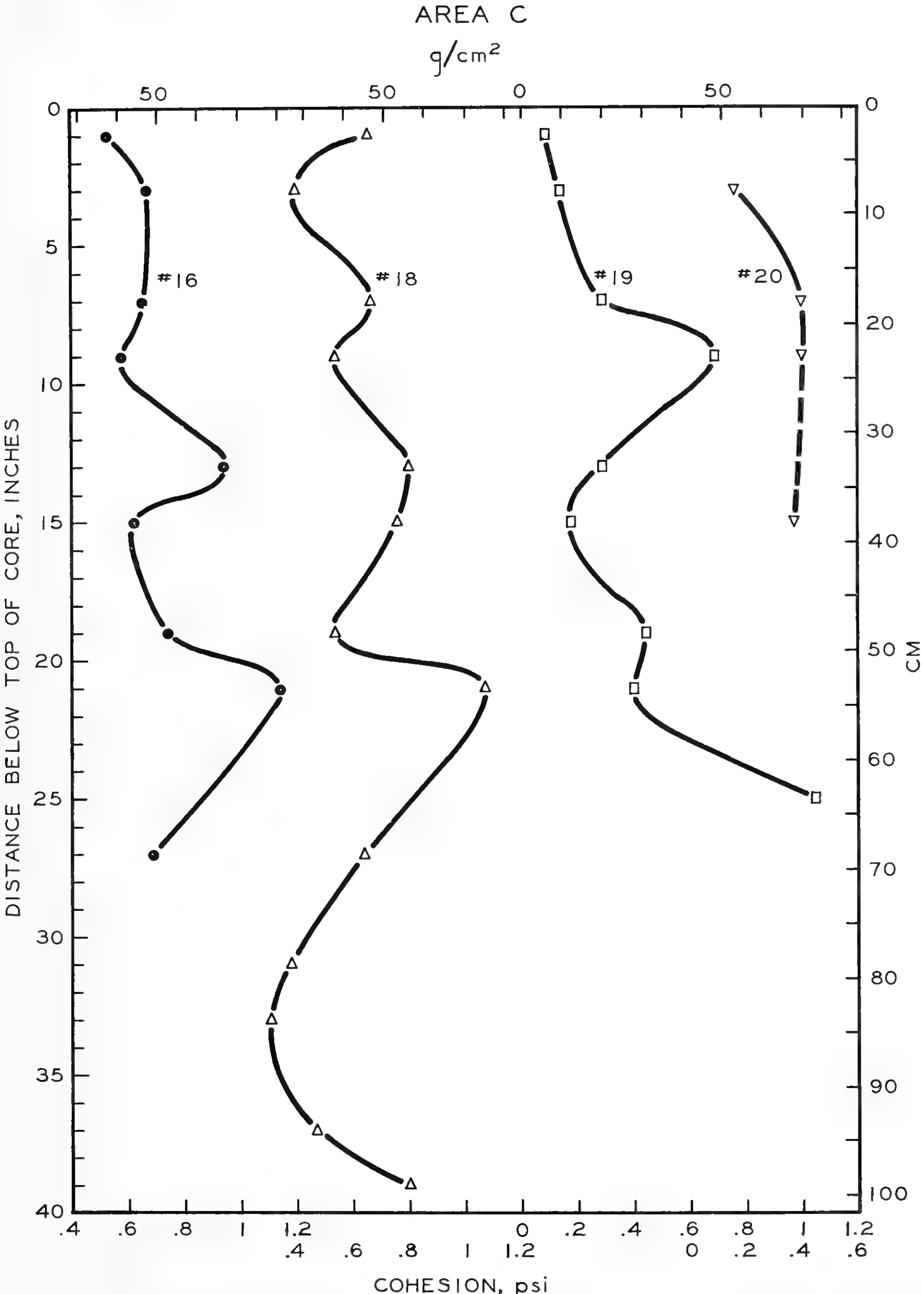


FIGURE 13. DEPTH RELATED TO COHESION (COMPRESSION TESTS): AREA C CORES

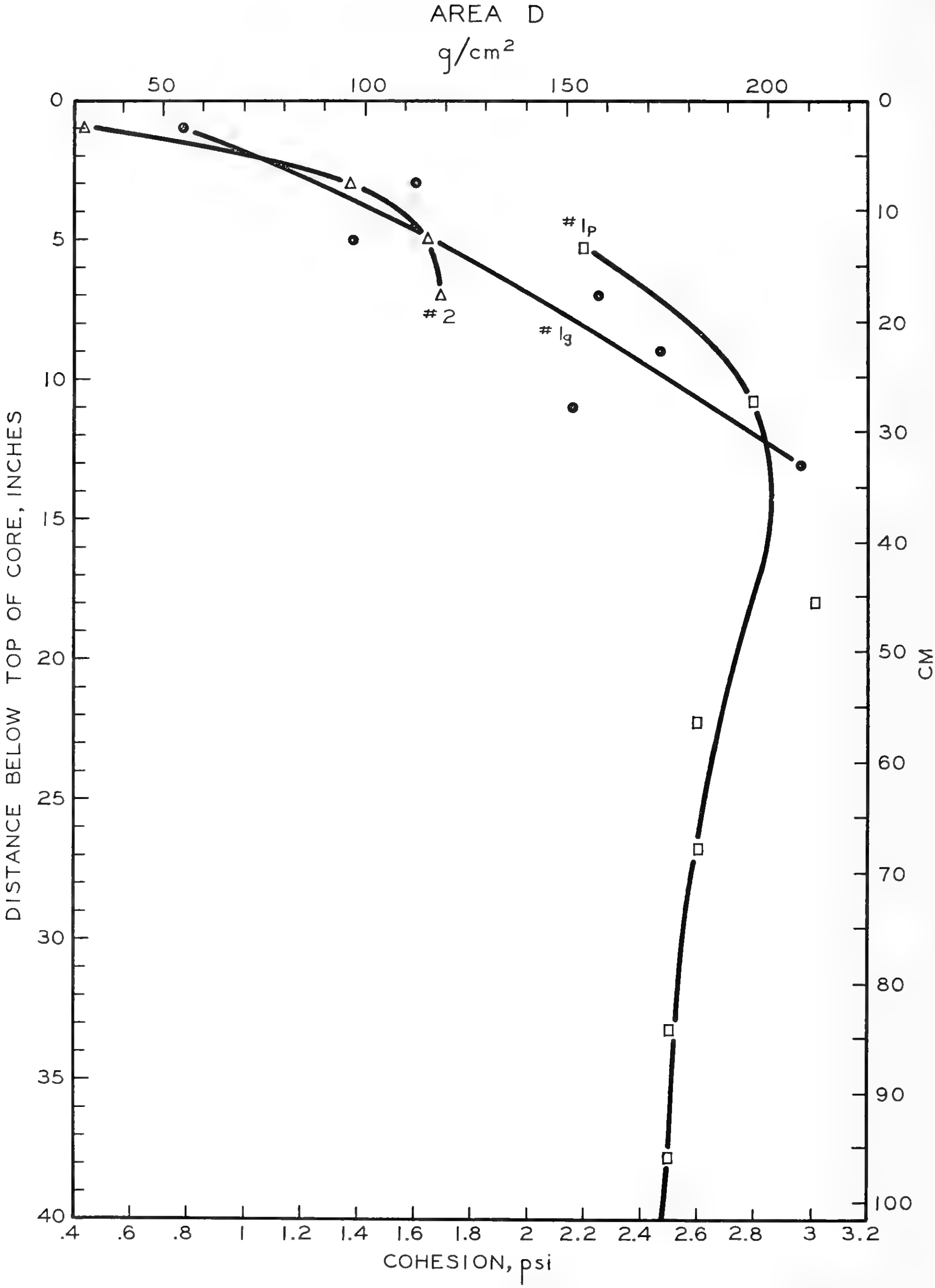


FIGURE 14. DEPTH RELATED TO COHESION (COMPRESSION TESTS): AREA D CORES

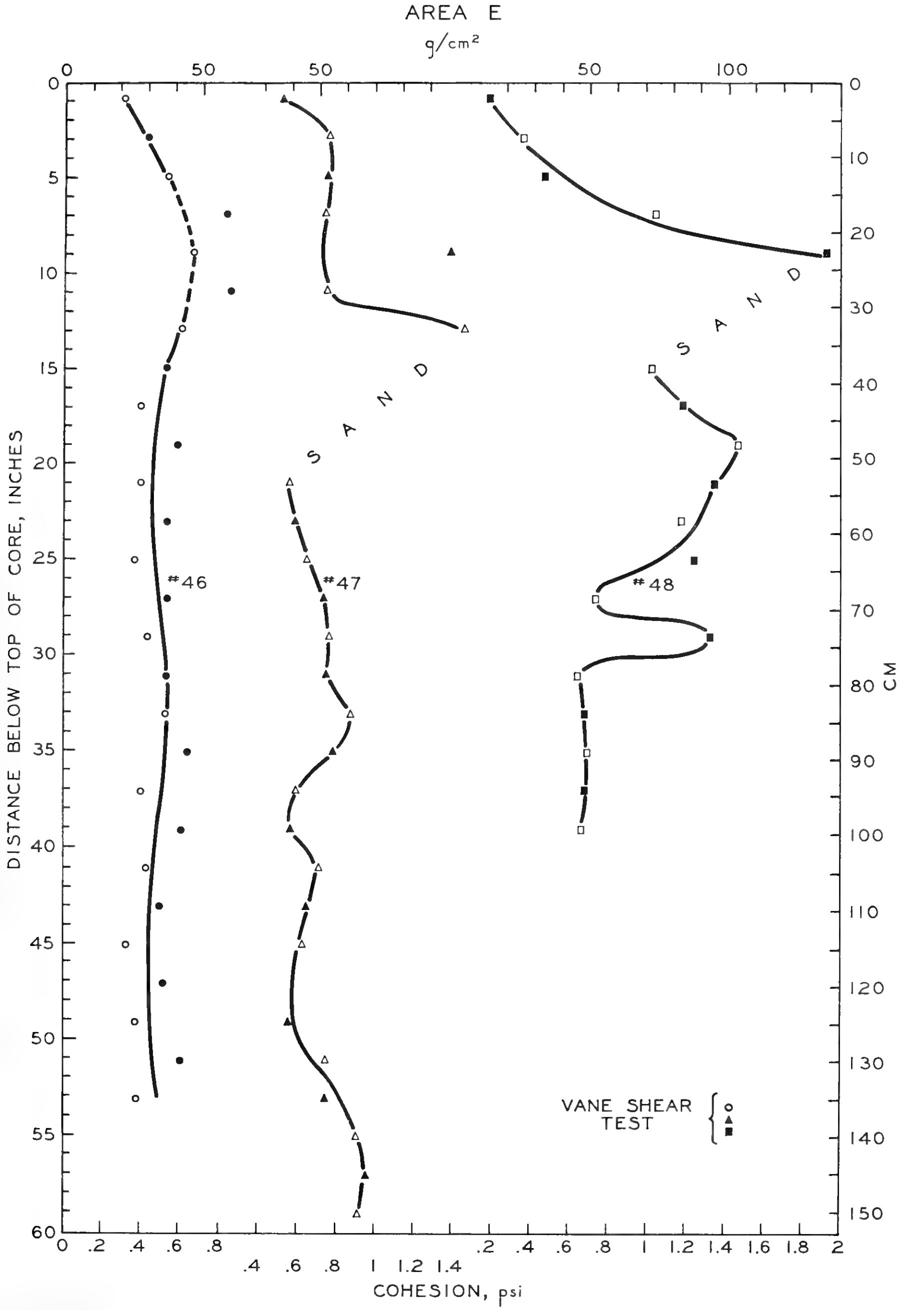


FIGURE 15. DEPTH RELATED TO COHESION (COMPRESSION AND VANE TESTS): AREA E CORES

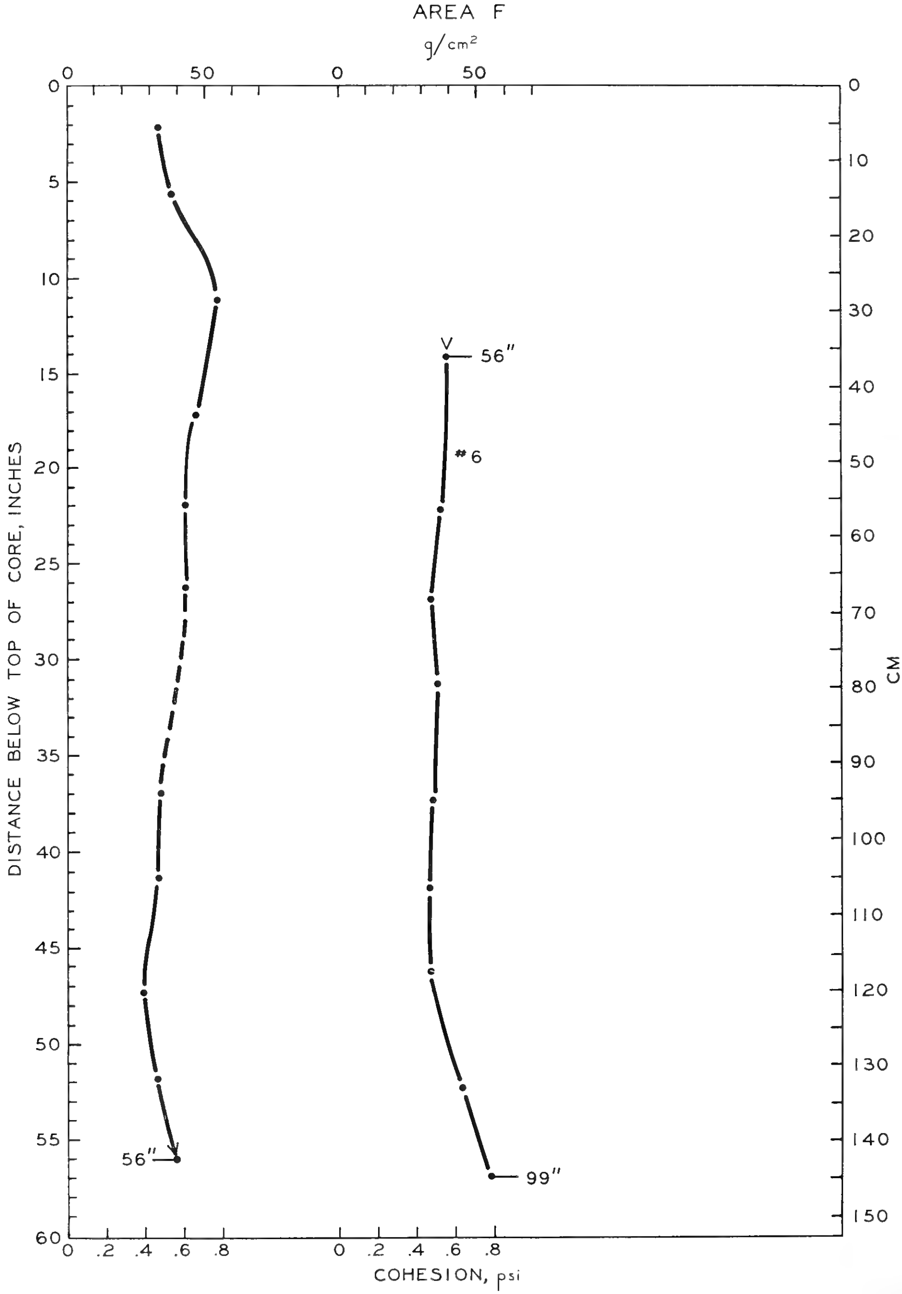


FIGURE 16. DEPTH RELATED TO COHESION (COMPRESSION TESTS): AREA F, CORE 6

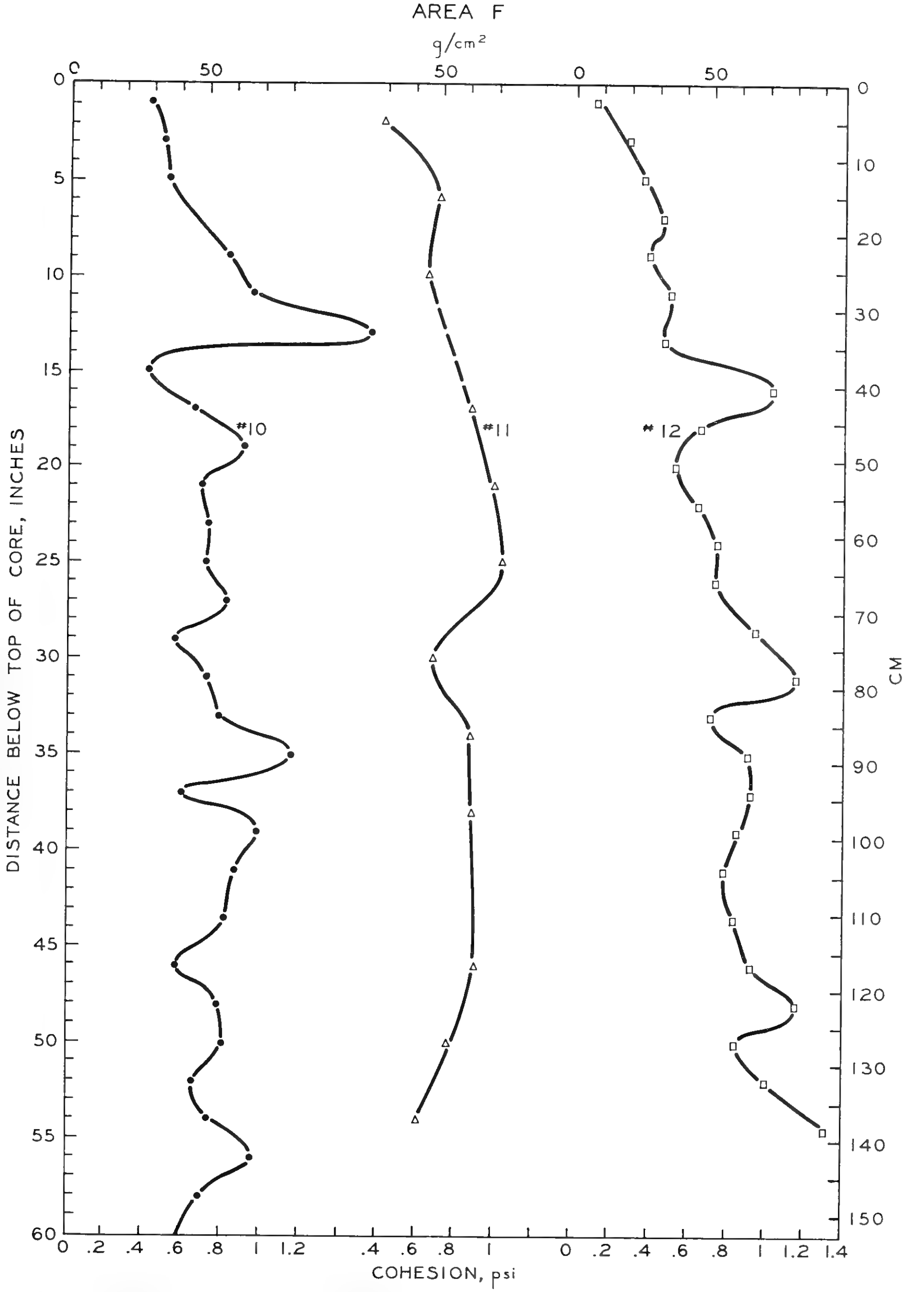


FIGURE 17. DEPTH RELATED TO COHESION (COMPRESSION TESTS):
AREA F, CORES 10-12

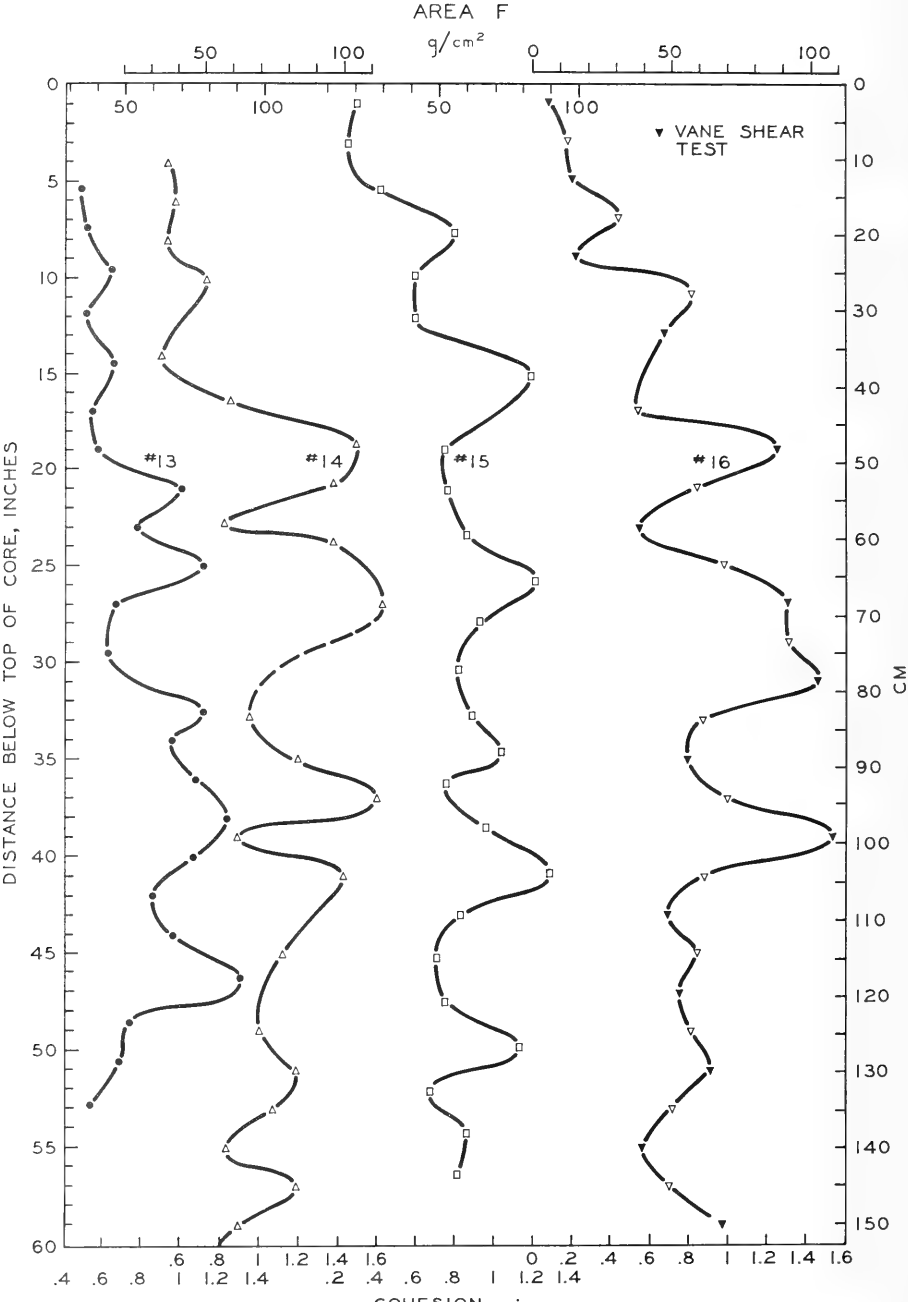


FIGURE 18. DEPTH RELATED TO COHESION (COMPRESSION AND VANE TESTS): AREA F, CORES 13-16

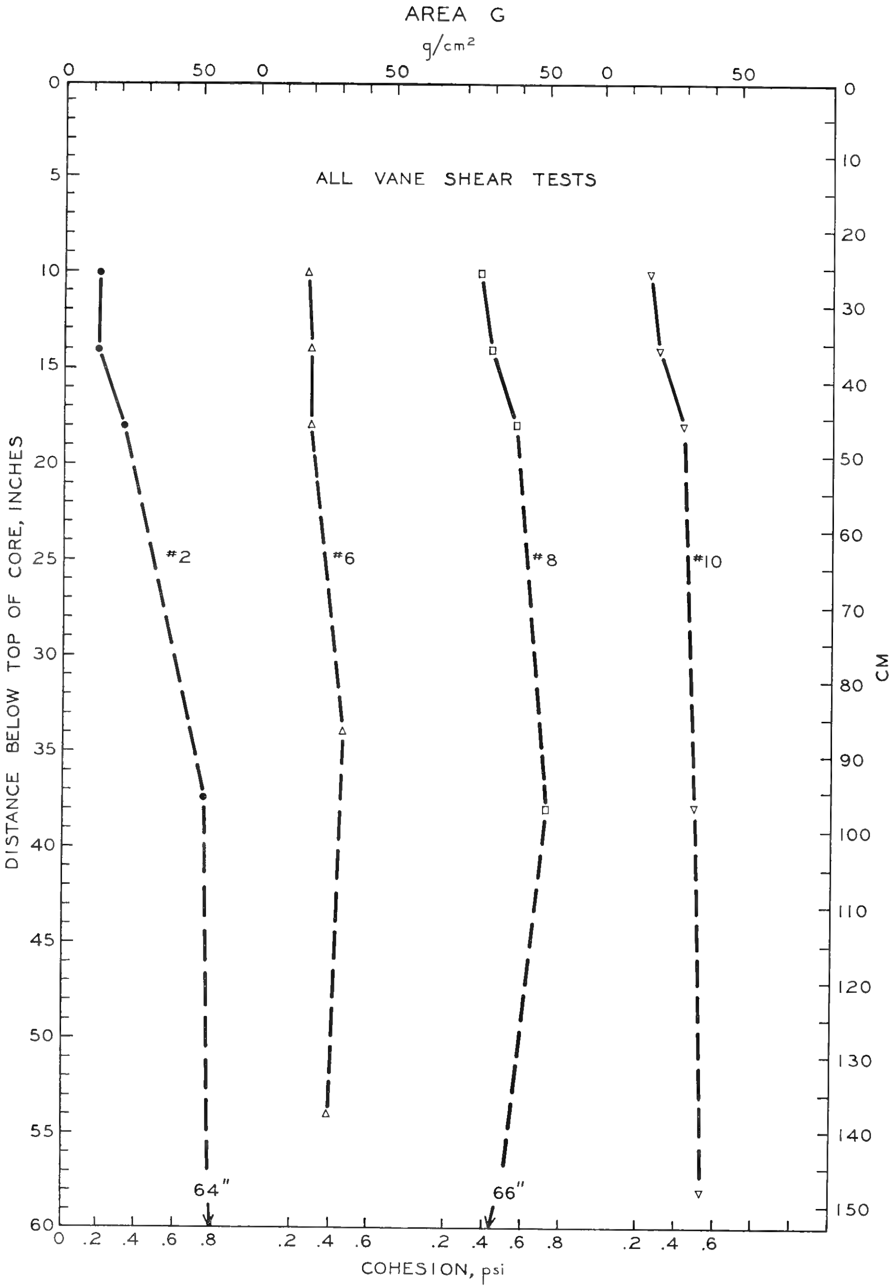


FIGURE 19. DEPTH RELATED TO COHESION (VANE TESTS): AREA G CORES

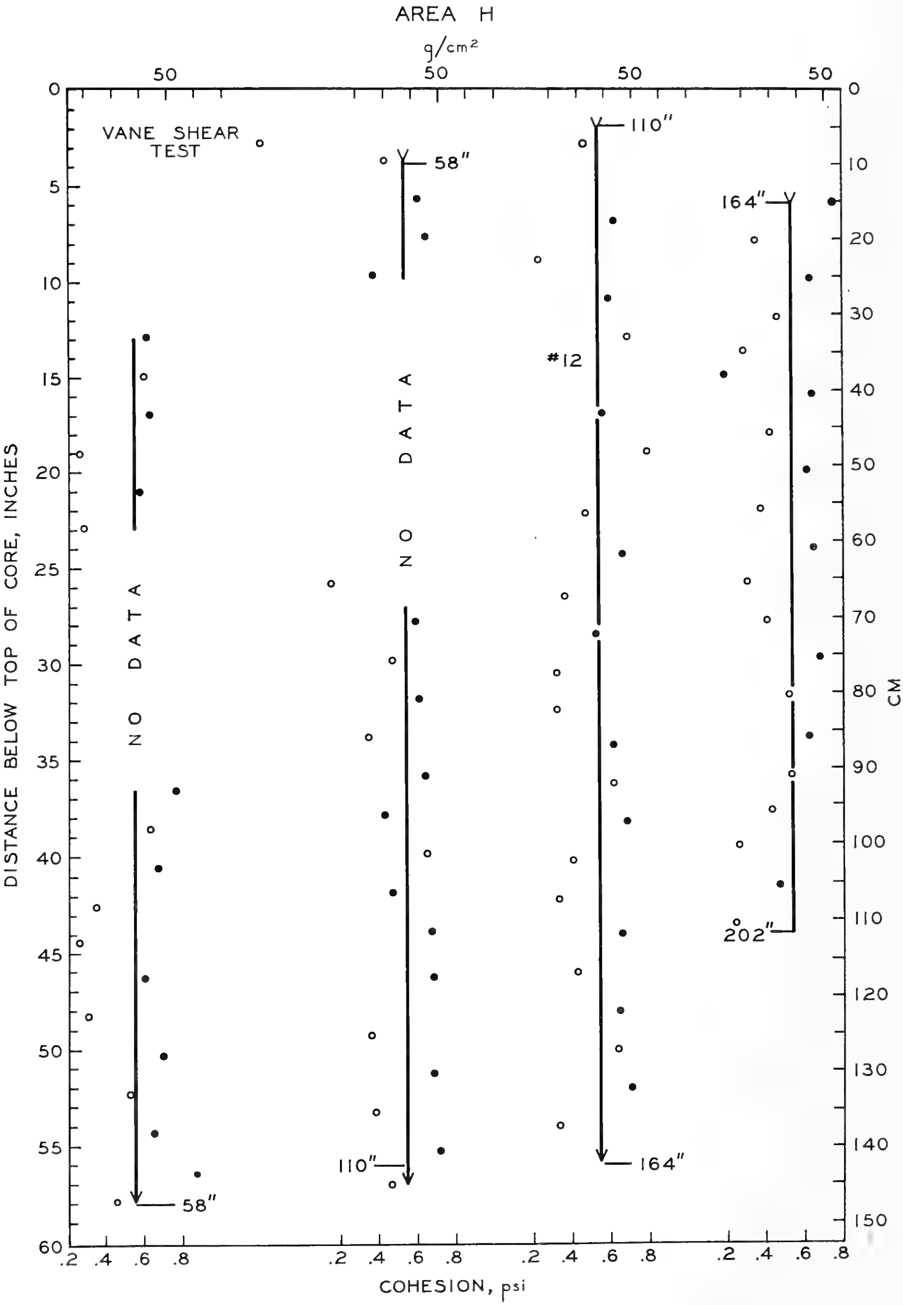


FIGURE 20. DEPTH RELATED TO COHESION (COMPRESSION AND VANE TESTS): AREA H, CORE 12

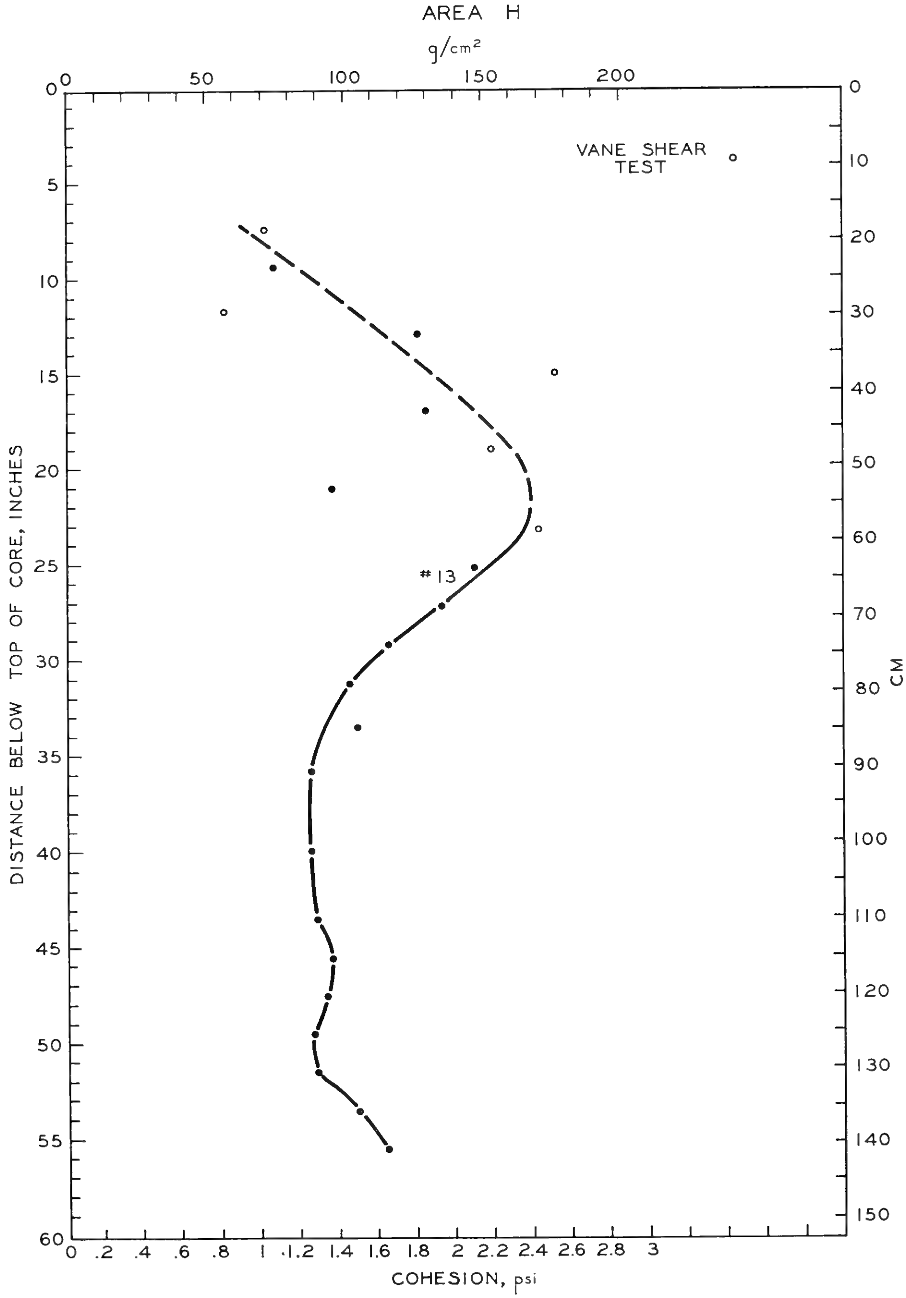


FIGURE 21. DEPTH RELATED TO COHESION (COMPRESSION AND VANE TESTS): AREA H, CORE 13

(Fig. 14). It is possible the apparently erratic cohesion values may be valid, but the probability is not considered great enough to warrant curve fitting to the points. On the other hand, curves of values from cores in Area C (Fig. 13) and most from Area F (Figs. 17 and 18) show multiple inflections that are considered valid. Here, graphs of water content versus distance from the tops of the core (Richards, in preparation) also show the same inflection pattern; it is not unreasonable to suppose that the in-place variation of strength and other physical properties is real.

The difference in the distribution of the strength profile in the two cores from Area H (Figs. 20 and 21) is perplexing, and all the more so because sediment sensitivity of the two cores is so nearly alike (Fig. 22). Core 13 is from the Kullenberg gravity corer used as the trip weight for piston core 12. The red or pelagic clay sediment in both cores should be essentially identical. Relative strength measurements on a pelagic clay core by Arrhenius (1952, plate 2.51), when converted (Fig. 23), show a distribution of values similar to those of the gravity core (Fig. 21), indicating that the latter values may be more valid than those from the piston core (Fig. 20).

In summary, this investigation shows that cohesion usually increases with increasing distance below the top of the core (Table 4). This increase may be relatively regular or, more often, irregular. A few cores possessed a strength distribution relatively uniform from top to bottom. None of the cores show a strength profile that decreases from top to bottom over the entire length, although a decrease may occur over one or more short distances. The least cohesion measured is about 4.2 g/cm^2 (0.06 psi) at the top of core A 23, and the greatest is 234 g/cm^2 (3.3 psi) in the section 66 to 71 cm (26 to 28 in) below the top of core B 83. Almost all cores show minimum cohesion at or near the top of the core (Table 4). Twice as many of the maximum values occur at some intermediate distance in the core rather than at the bottom. I show (in preparation) that cohesion in these cores is inversely related to water content and a variable relation to grain size. Maximum cohesion at intermediate depths, however, is usually related to a coarser grain size and a lower water content.

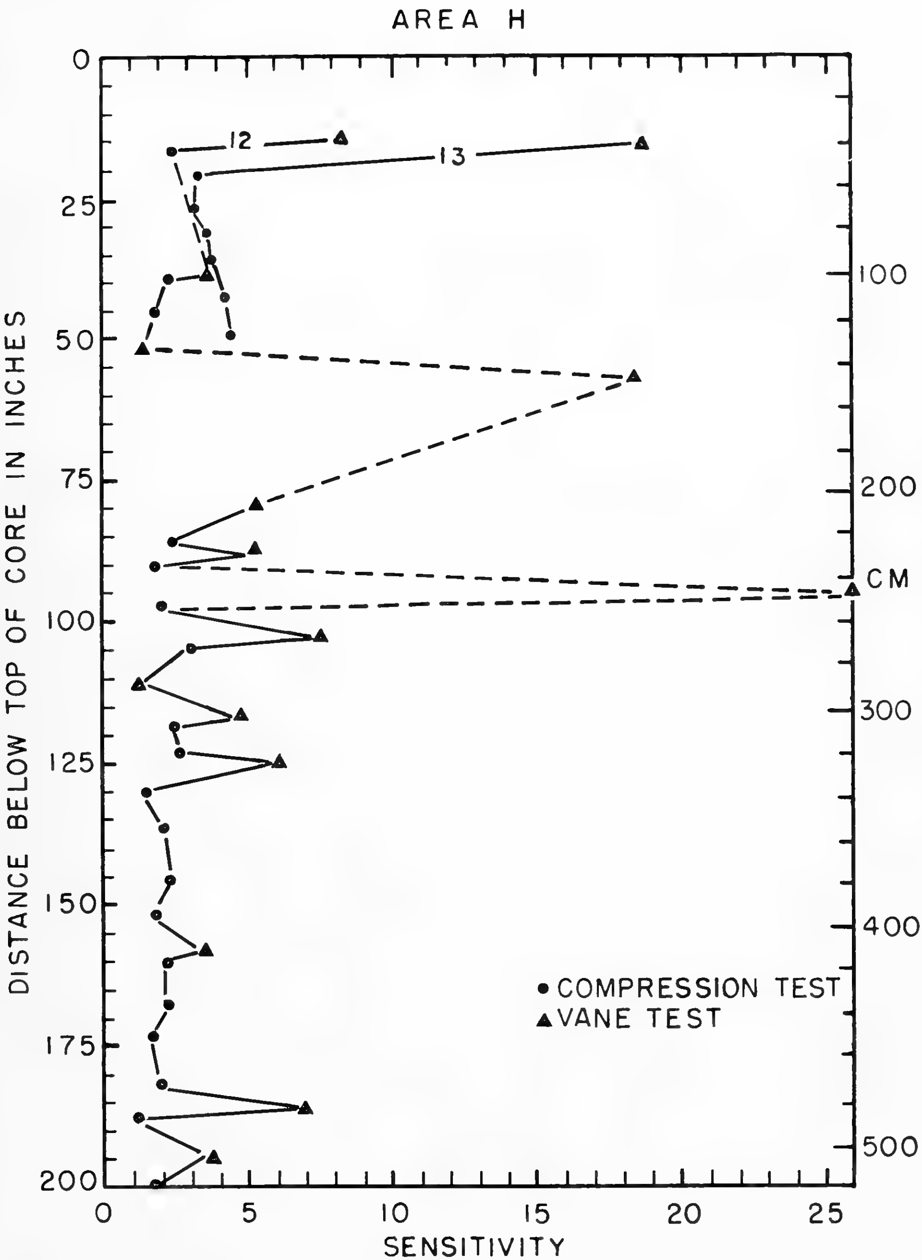


FIGURE 22. SEDIMENT SENSITIVITY FOR AREA H CORES

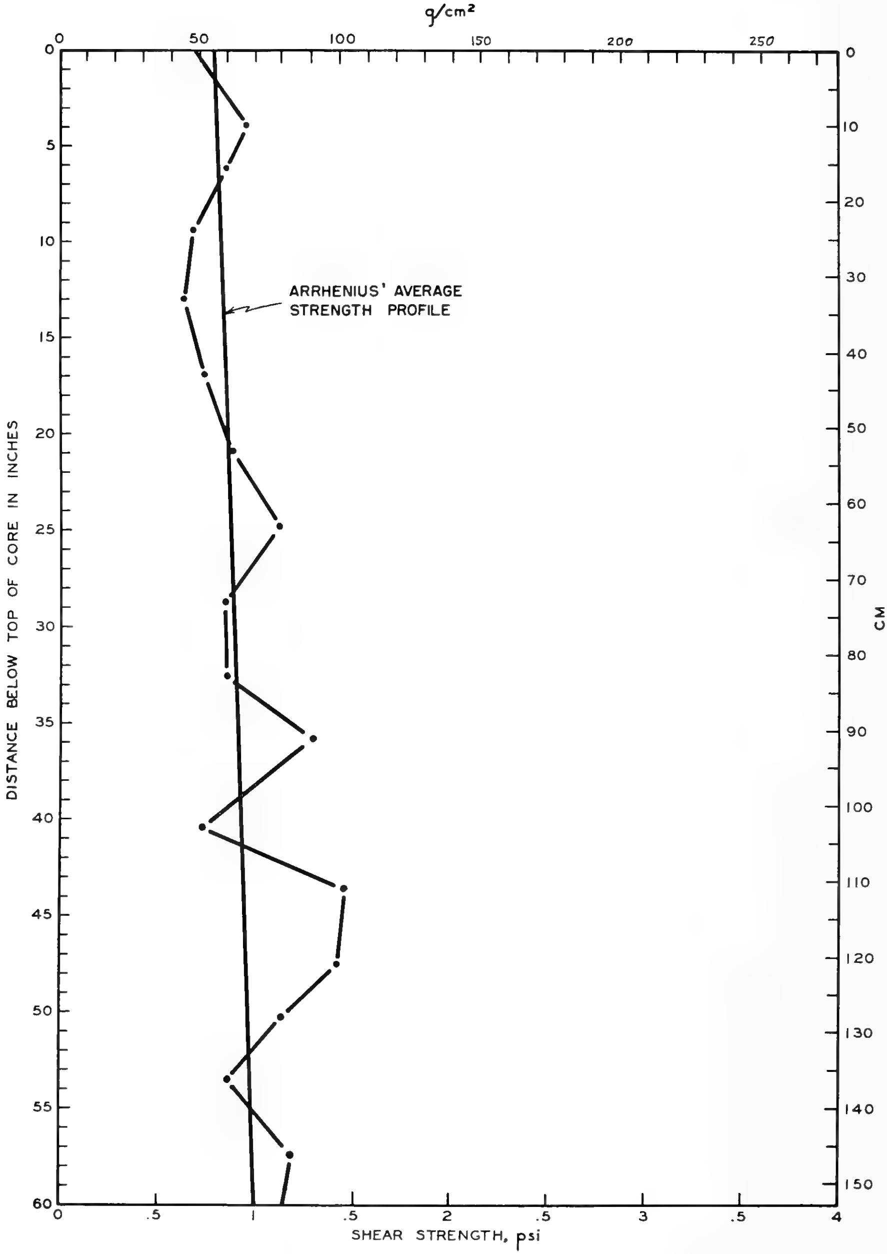


FIGURE 23. SHEAR STRENGTH IN THE UPPER PART OF A PELAGIC CLAY CORE FROM THE EAST PACIFIC OCEAN. (After Arrhenius, 1952, Appendix pl. 2.51, with conversion of relative strength values after Moore and Richards, submitted)

TABLE 4. SUMMARY OF COHESION DISTRIBUTION IN CORES

Core No.	Type Core	Average Cohesion Profile vs. Depth[1]	Cohesion Distribution				Core Length Tested (in)
			Minimum (g/cm^2)	Depth (in)	Maximum (g/cm^2)	Depth (in)	
A 23	Gravity	Regular Increase	4	0-2	37	24-26	28
A 31	Gravity	Regular Increase	9	2-4	64	32-34 & 38-40	40
A 33	Gravity	Regular Increase	8	0-2	29	32-34	34
B 83	Gravity	Irregular Increase	23	2-4	234	26-28	36
B 85	Gravity	Regular Increase	19	2-4	100	24-26	28
B 87	Gravity	Irregular Increase	26	0-2	97	20-22	28
C 16	Piston	Irregular Increase	37	0-2	81	20-22	28
C 18	Piston	Irregular Increase	22	32-34	75	20-22	40
C 19	Piston	Irregular Increase	6	0-2	73	24-26	26
C 20	Piston	Regular Increase	<11	2-4	28	6-10	16
D 1g	Gravity	Regular Increase	55	0-2	208	12-14	14
D 2	Gravity	Regular Increase	31	0-2	119	6-8	8
D 1p	Piston	Relatively Uniform[2]	77	4.5-9	105	15.5-20	175
E 46	Gravity	Uniform	22	0-2	47	8-10	54
E 47	Gravity	Relatively Uniform	40	20-22	103	12-14	60
E 48	Gravity	Irregular Increase	13	0-2	136	8-10	40

1/ Depths are given in inches to facilitate location in the Figures.

2/ Only top 40 inches graphed (Fig. 14); cohesion about 90 g/cm^2 in remaining core length.

TABLE 4. SUMMARY OF COHESION DISTRIBUTION IN CORES (Cont'd)

Core No.	Type Core	Average Cohesion Profile vs. Depth[1]	Cohesion Distribution Minimum (g/cm^2)	Depth (in)	Maximum (g/cm^2)	Depth (in)	Core Length Tested (in)
F 6	Gravity	Relatively Uniform	27	45–49.5	55	9–13.5 & 96.5–101	101
F 10	Gravity	Irregular Uniform	29	0–2	110	12–14	65[3]
F 11	Gravity	Irregular Increase	29	0–4	73	23–27	56
F 12	Gravity	Irregular Increase	8	0–2	93	53–56	56
F 13	Gravity	Irregular Increase	34	4.5–6.5[4]	91	45–47.5	54
F 14	Gravity	Irregular Increase	37	3.2–5.2[4]	114	26–28	64[3]
F 15	Gravity	Irregular Increase	18	2.2–4.2	91	39.8–42	59
F 16	Gravity	Irregular Increase	5	0–2	101	38–40	62[3]
G 2[5]	Piston	Regular Increase	12		53		66[3]
G 6[5]	Gravity	Relatively Uniform	18		31		56
G 8[5]	Gravity	Regular Increase	25		52		68[3]
G 10[5]	Gravity	Regular Increase	17		36		60
H 12	Piston	Uniform		av. 38 g/cm^2			202.5
H 13	Gravity	Irregular Uniform	72	6.5–8.5	177	14–16	56.5

[3] Depth below 60 inches not graphed.

[4] No tests at lesser depths.

[5] Too few test data to warrant showing minimum and maximum depth of cohesion values in cores.

IV. VALIDITY OF STRENGTH DATA

Disturbance to natural, in place, sediment may result in micro-structural damage with consequent loss of in-place strength. The amount of damage generally is proportional to the grain size. Compared to sediments composed of sand- and silty sand-size particles that possess little or no cohesion, cohesive, fine-grained sediments are relatively immune to damage if they possess a low sensitivity or a low water content; however, sediments of high sensitivity or high water content are very susceptable to disturbance.

A difficult question to answer is how valid are shear-strength measurements considering the various sources of sedimentary macro- and micro-structural disturbance that occur between the time sediment was in place on the sea floor and the time of the test? These sources of disturbance include: (1) collection of the sediment core by the sampler, particularly gravity-type corers, (2) reduction of hydrostatic pressure from in place to atmospheric pressure and its effect on the differential expansion of interstitial water and mineral grains, (3) shipboard handling and storage of the core in liner or barrel, including bacterial and chemical changes occurring during the storage period, (4) transportation to the laboratory, and (5) test procedure in the laboratory. Hvorslev (1949, pp. 182-206) presents an excellent detailed discussion or many factors contributing to disturbance.

An unknown but presumably small amount of structural disturbance results from the slight expansion of water and grains with a reduction of confining pressure (about one decibar or 1.47 psi per meter of sea-water depth). With care, given to most of the cores described, it is believed that disturbance caused by handling and shipment was reduced to a small or negligible amount. In the laboratory, a sample can be tested within the core liner without extrusion by using a vane shear machine, thus nearly eliminating disturbance from shear-test procedure. Loss of strength resulting from extrusion for the compression and vane shear tests or cores described in this report is presumed to be present, but of small quantity. Some of the strength lost by the various processes may be regained by thixotropy[3] following core collection and prior to testing. Studies by Skempton and Northey (1952, pp. 33-39) indicate that because of the short time involved, the amount of strength regain probably would be small. Without doubt, if due care has been exercised elsewhere, the major source of disturbance and loss of strength is in the sampling process. Jakobson (1954, p. 56) concluded from a series of tests, designed to provide information on the influence of sampler type on shear strength of clay-size samples, that even the best type of sampler

[3] Thixotropy is the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible (ASCE).

caused a 10 percent reduction in shear strength compared to in-place tests. Improved piston samplers carefully designed in light of recommendation by Hvorslev (1949) have proven themselves in Norway (Bjerrum, 1954a, p. 55; Vold, 1956) and in Sweden (Kallstenius, 1958, p. 72-73), as previously mentioned. Initial success with the Hydroplastic corer (Richards and Keller, in press) suggests that better engineered gravity and piston corers may provide a partial solution in submarine investigations. A still better solution is in-place strength testing. This already has been accomplished in the shallow-water, marine environment (Moore, personal communication), but in-place testing in deep water has yet to be attempted.

One means of estimating the likelihood of disturbance is by measurement of sensitivity; the greater the sensitivity the larger the loss of strength in the completely disturbed or remolded condition. Skempton and Northey (1952, p. 31) have classified clay-size (silt-size particles also show sensitivity, p. 33) sediments according to sensitivity. Their classification has been extended by Rosenqvist (1953, p. 195), whose system is shown in Table 5, together with the percentage loss of strength upon remolding an "undisturbed" sample computed by me.

Sensitivities of core samples are given in Table 6 and Figure 22. Most values are in the range of 2 to 6, medium to very sensitive, while three from Area H have sensitivities between 17 and 26, indicating that the samples tested are medium quick. (The word "quick" in soil mechanics denotes a fluid consistency in the remolded state without much strength). Sensitivities resulting from vane tests in Core H 12 (Fig. 20) are higher than those from compression tests as a result of lower values of remolded cohesion measured by the vane tests. Although the in-place sensitivities of deep-sea sediments are unknown, the presence of medium to very sensitive sediments, of medium to high water content (Richards, in preparation), in most of the cores is an indication that in-place sediments may be even more sensitive; more loss of strength may have occurred in sampling than suspected.

Although values of shear strength presented are conservative by an unknown amount compared to in-place strength, because a quantitative estimate of the reduction of in-place strength cannot be made with the information available, values reported are considered sufficiently reliable for engineering use at the present time.

TABLE 5. SENSITIVITIES OF FINE-GRAINED SEDIMENTS

Sensitivity	Description	Percentage of "Undisturbed" Strength Lost in Remolded State
ca 1	Insensitive	0
1 - 2	Slightly insensitive	0 to 50
2 - 4	Medium sensitive	50 to 75
4 - 8	Very sensitive	75 to 87.5
8 - 16	Slightly quick	87.5 to 93.8
16 - 32	Medium quick	93.8 to 96.9
32 - 64	Very quick	96.9 to 98.4
> 64	Extra quick	> 98.4

TABLE 6. SENSITIVITY OF SEDIMENTS

Core No.	Compression or Vane Test	Sensitivity	Location of Sample in Core (in)
A 31	c	1.6	8-10
B 85	c	2.2	14-16
	c	2.9	26-28
B 87	c	3.0	12-14
C 16	c	3.1	8-10
	c	3.4	20-22
C 18	c	1.7	14-16
	c	2.0	32-34
C 19	c	1.9	14-16
	c	2.2	24-26
D 1p	c	2.2	4.5-9
	c	2.8	35.5-40
E 46	c	2.6	12-14
	v	2.7	16-18
	v	2.2	28-30
	c	3.6	30-32
	v	2.5	44-46
	c	2.6	46-48

TABLE 6. SENSITIVITY OF SEDIMENTS (Cont'd)

Core No.	Compression or Vane Test	Sensitivity	Location of Sample in Core (in)
E 47	v	3.9	8-10
	v	5.0	26-28
	v	3.6	48-50
F 12	c	2.6	17-19
	c	4.2	36-38
	c	4.1	51-53
F 13	c	4.0	10.8-13
	c	4.7	18-20
	c	3.4	33-35
	c	3.5	39-41
	c	4.5	45-47.5
	c	2.6	51.5-53.8
F 14	c	3.4	5.2-7.2
	c	3.7	23.8-26
	c	2.8	48-50
	c	4.7	60-62
G 2	v	6.1	8-12
	v	3.2	12-16
	v	4.5	16-20
	v	4.2	35.4-39.4
	v	4.1	62.5-66.5
G 6	v	4.1	8-12
	v	3.8	12-16
	v	4.7	16-20
	v	4.9	32-36
	v	3.4	52-56
G 8	v	6.2	8-12
	v	4.7	12-16
	v	5.6	16-20
	v	5.8	36-40
	v	3.8	64-68
G 10	v	6.1	8-12
	v	5.1	12-16
	v	4.4	16-20
	v	3.7	36-40
	v	4.1	56-60

TABLE 6. SENSITIVITY OF SEDIMENTS (Cont'd)

Core No.	Compression or Vane Test	Sensitivity	Location of Sample in Core (in)
H 12	v	8.2	14-16
	c	2.5	16-18
	v	3.8	37.5-39.5
	c	2.4	39.5-41.5
	c	2.0	45.2-47.2
	v	1.6	51.2-53.2
	v	18.5	56.8-58.8
	v	5.4	78.8-80.8
	c	2.4	84.8-86.8
	v	5.4	86.8-88.8
	c	1.9	88.8-90.8
	v	25.9	94.8-96.8
	c	2.1	96.8-98.8
	v	7.5	102.2-104.2
	c	3.0	104.2-106.2
	v	1.4	110.2-112
	v	4.7	116-118
	c	2.6	118-120
	c	2.8	122-124
	v	6.2	124-126
	v	1.4	129.2-131.2
	c	2.2	135.5-137.5
	c	2.4	145.2-147.2
	c	1.7	151-153
	v	3.5	157-159
	c	2.2	159-161
	c	2.2	167-169
	c	1.7	173-175
	c	2.0	181-183
	v	7.0	184.8-186.8
	c	2.0	186.8-188.8
	v	3.7	192.8-194.8
	c	1.7	198.5-200.5
H 13	v	18.0	14-16
	c	3.4	20-22
	c	4.2	26.2-28.2
	c	4.8	30.2-32.2

TABLE 6. SENSITIVITY OF SEDIMENTS (Cont'd)

Core No.	Compression or Vane Test	Sensitivity	Location of Sample in Core (In)
H 13	c	4.9	34.8-36.8
	c	5.4	42.5-44.5
	c	5.6	48.5-50.5

V. PRACTICAL APPLICATION OF DATA TO STRUCTURAL LOADS PLACED ON THE SEA FLOOR

A. GENERAL STATEMENT AND SIMPLE STRENGTH THEORY

If an object is placed on sediments of the sea floor without impact velocity, one of two events will immediately occur: (1) either the sediment will have insufficient strength and will fail in shear (rupture) or, (2) the sediment will have sufficient strength to support the object, which will remain on the surface. Following either of these events, a relatively slow and gradual settlement of the object may occur because of sedimentary consolidation.

Whether or not the object initially sinks, or penetrates, is directly related to the structural strength of the sediment, the amount of load, and can be determined from a knowledge of the sediment bearing capacity. The assumption that an object will penetrate until it is supported by sediment with a wet density equal to or greater than that of the object is invalid (see Jürgenson, 1934, p. 199). Archimedes' principle, which states that a body immersed in a fluid is buoyed up with a force equal to the weight of the displaced fluid, does not apply because sediment has strength and is not a fluid. This strength, however, may be very small if the material is fine-grained and has a high water content. (The fact that an object placed in water is buoyed up by the unit weight of water should not be confused with these statements. The effective mass of the load on the sediment is its buoyed weight).

A simplified expression of shear strength in sediments has been developed from a theory presented by Coulomb (1776)

$$s = c + p \tan \phi \qquad (8)$$

where s is the shear strength, c is the cohesion, p is the total intergranular pressure normal to the shear plane, and ϕ is the angle of internal friction. Results of investigations conducted by Hvorslev (1936, 1937) showed that cohesion is a function of water content, and Coulomb's equation should be modified to include an expression of the effective normal stress on the shear plane. The earlier discovery of effective stress by Terzaghi, and later experimental confirmation by Rendulic and others, is discussed in detail by Skempton (1960) and need not be considered here.

Equation 8 is usually expressed in the form

$$s = c + (\sigma - u) \tan \phi \qquad (9)$$

where s is the shear strength, c is the apparent cohesion, σ is the total stress normal to the shear plane, u is the neutral stress (pore pressure) at the point on this surface,

$(\sigma - u)$ or $\bar{\sigma}$ is the effective stress (intergranular pressure) at this point, and ϕ is the angle of internal friction or, more properly according to Terzaghi (1943, p. 7), angle of shearing resistance. Bjerrum (1954b, p. 3-9) presents a more detailed discussion.

B. ULTIMATE BEARING CAPACITY

The sediments investigated are water-saturated (Richards, in preparation) and largely cohesive. Cohesive sediments contain significant amounts of fine-grained, silty clay- and clay-size particles that possess plasticity. (Plasticity is the property of sediment that allows it to be deformed beyond the point of recovery without cracking or appreciable volume change--ASCE). The problem of determining bearing capacity of water-saturated, cohesive sediments has been considered relatively simple, although recent investigations at the Massachusetts Institute of Technology indicate that a number of simplifying assumptions are questionable (Lambe, 1961; Whitman, 1961). Saturated clayey sediments stressed without change in water content behave with respect to applied stress as if they were purely cohesive materials with an angle of shearing resistance equal to zero ($\phi = 0$), the shear strength, as well as the effective stress, being independent of the total stress on the failure plane. In this special instance, equation 9 becomes

$$s = c \tag{10}$$

Bjerrum and Kjaernsli (1957, p. 2-3) have presented a short summary of the development of the $\phi = 0$ analysis by Skempton and others, which currently enjoys wide usage in soil mechanics.

Ultimate bearing capacity is reached just before sediment fails in shear under an applied load. It is related to the product of the shear strength and one or more factors, which are functions of the size, shape, and depth of loading. Bearing capacity of plastic clays is nearly independent of the loaded surface area (Terzaghi, 1925, p. 1065).

A formula for ultimate bearing capacity of sediment under strip loading (load infinitely long relative to the width), as developed from equations by Prandtl (1920) and modified by Terzaghi (1943, p. 118-134), is

$$q_o = c\,N_c + \gamma d\,N_q + \gamma b\,N_\gamma \tag{11}$$

where q_o is the ultimate bearing capacity (the average load per unit area required to produce failure by rupture of a supporting sediment mass), γ is the effective unit weight, b is the half width of the strip load, d is the depth of the load below the surface, and N_c, N_q, and N_γ are dimensionless constants dependent on ϕ.

For surface loading and when ϕ is assumed to be zero

$$q_o = c\ N_c \tag{12}$$

where N_c is $(\pi + 2)$ or 5.14 according to Prandtl. For loading in which the length to width ratio is less than 2.0 (square or circular loads) equation 12 usually is modified to

$$q_o = 1.3\ c\ N_c \tag{13}$$

Other investigators believe that N_c constants 11 to 12 percent higher should be used for strip, square, and circular loads at the surface. Different N_c values must be used below the surface (Fig. 24, after Skempton, 1951, p. 181).

An example[4] using data from core A 31 will illustrate the use of one of these formulas. A load with a buoyed mass of 5000 g and a square surface area of 100 cm^2 is placed on the bottom in the immediate vicinity of core A 31 without impact velocity. The resulting pressure or stress on the sediment is 5000/100 or 50 g/cm^2. An ultimate bearing capacity of at least the same amount is required for support. Assuming surface loading, $q_o = 6.7\ c$ (equation 13), the cohesion necessary for support is 7.5 g/cm^2 (0.11 psi). In this core (Fig. 11), the sample from 0 to 5 cm (0 to 2 in) was too soft to test. The sample from 5 to 10 cm has a measured cohesion of 9.1 g/cm^2 (0.13 psi). Interpolation of the strength profile indicates shear failure of the sediment at a depth of somewhat less than 8 cm (3 in). Allowing a safety factor of 1.5 (the more usual value of 3, according to Meyerhof, 1951, p. 301, is more applicable to sediment with a higher sensitivity), a cohesion of 10 g/cm^2 (about 0.14 psi) is required for adequate support and is found at an interpolated depth of about 8 cm.

It is important to realize that the strength profile-depth relation determined in a single core may not be valid a short distance away from where the core was collected unless supplemented with tests of other nearby samples. Almost no information exists on the areal variability of strength in marine sediments.

C. CONSOLIDATION AND SETTLEMENT

The gradual reduction in volume of a sediment mass resulting from an increase in compressive stress is termed consolidation (ASCE). If an object placed on the bottom produces this stress due to its unit load, consolidation will occur and the object will

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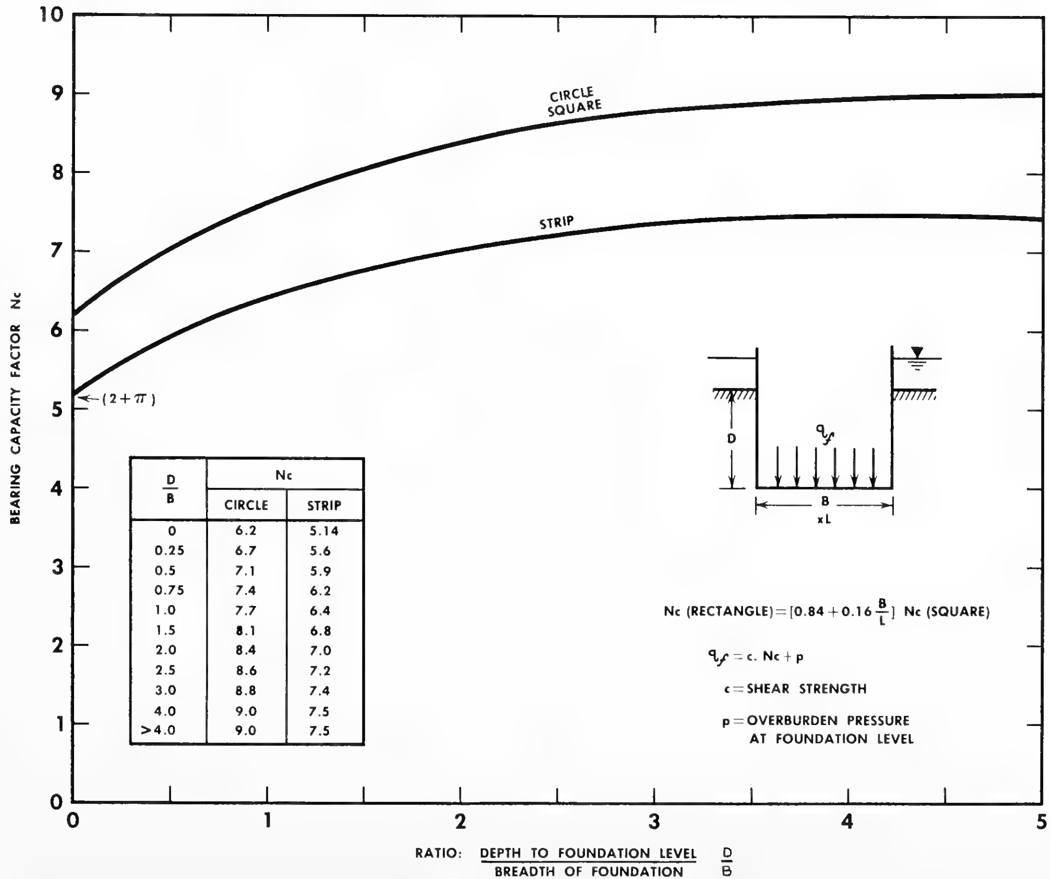
[4]This, and the following examples, were selected to show methods of calculation; they are not necessarily to be considered practical engineering problems.



$\frac{D}{B}$	Nc	
	CIRCLE	STRIP
0	6.2	5.14
0.25	6.7	5.6
0.5	7.1	5.9
0.75	7.4	6.2
1.0	7.7	6.4
1.5	8.1	6.8
2.0	8.4	7.0
2.5	8.6	7.2
3.0	8.8	7.4
4.0	9.0	7.5
>4.0	9.0	7.5

FIGURE 24. BEARING CAPACITY FACTORS FOR OBJECTS PLACED ON CLAY-SIZE MATERIAL IN WHICH $\phi = 0$
(After Skempton, 1951, p. 181)

gradually settle as transference of load from water to solids is accompanied by a reduction in the volume of sediment equal to the volume of interstitial or pore water drained. Determination of the expected amount and time rate of settlement is made from results of laboratory consolidation tests.

<u>Amount</u> -- The amount of settlement is computed from the formula

$$\text{Settlement} = \frac{e_i - e_f}{1 + e_i} H_i \tag{14}$$

where e_i is the initial average void ratio and e_f is the final average void ratio within a sediment stratum undergoing consolidation with an initial thickness of H_i.

A problem previously was worked using data from core A 31. It was found that a hypothetical load (with a buoyed mass of 5 kg and a square surface area of 100 cm^2) placed on the bottom immediately sank about 8 cm because the sediment failed in shear. We will continue the problem at this point and compute the amount of expected settlement of the load due to sedimentary consolidation.

Step (1): compute depth in the core as a function of sediment overburden pressure (stress), d-p relationship (Fig. 25, curve A), from known values of sediment wet unit weight (Table 7).

TABLE 7. D-P COMPUTATION FROM WET UNIT WEIGHT OF CORE A 31

Interval (cm)	Interval Depth, d (cm)	Cumulative Depth (cm)	Wet Unit Weight, ρ 1/ (g/cm^3)	Overburden Stress $p = (\rho - 1)d$ (g/cm^2)	Cumulative Overburden Stress 2/ (g/cm^2)
0-4	4	4	1.4	1.6	1.6
4-15	11	15	1.45	5.0	6.6
15-29	14	29	1.5	7.0	13.6
29-46	17	46	1.55	9.4	23.0
46-60	14	60	1.6	8.4	31.4

1/ From measurements reported by Richards (in preparation).

2/ Plotted as curve A, Figure 3.

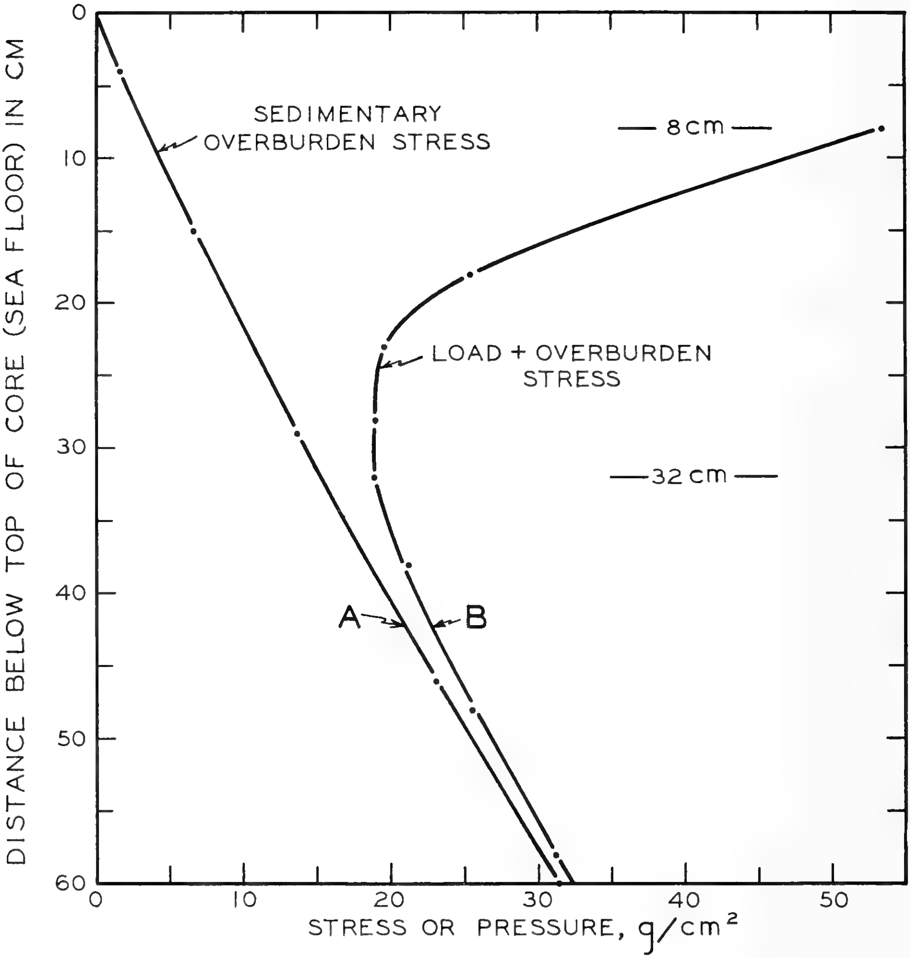


FIGURE 25. DEPTH RELATED TO STRESS IN AN EXAMPLE OF SETTLEMENT FOR CORE A 31

Step (2): compute the sum of the natural overburden stress and the computed pressure exerted by the load (Table 8). This computation of pressure in the direction of the vertical or z axis, p_z, of the load can be simplified by dividing the surface area into four rectangles (squares in the example given) to determine the pressure at the corner of each rectangle (see Fig. 26). A graphical solution (Fig. 26), from tables by Newmark (1935), is made for p_z, using coefficients m and n that are defined in Table 8 and Figure 26 (after Palmer, 1953, p. 39).

Step (3): load plus overburden stress from the last column in Table 8 is plotted (Fig. 25, curve B). Settlement is assumed negligible when curve B becomes asymptotic to curve A; 32 cm is selected as the lower limit of the compressible stratum in this problem (Fig. 25).

Step (4): an average initial void ratio must next be computed. Depth as a function of void ratio for core A 31 is shown in Figure 27. Curve B arbitrarily is divided into 5-cm intervals from 8 to 32 cm (Table 9). A void ratio is determined every 5th cm from Figure 27, and an average initial void ratio[5] of 2.41 is computed (Table 9).

Step (5): compute the average final void ratio. Values of load plus overburden stress at each depth given in Table 9 are determined from curve B, averaged, and an average final stress of 29 g/cm^2 is computed in Table 9. When this figure is entered on the laboratory-determined virgin compression curve of Figure 10, it is found that the average final void ratio is 2.08.

Step (6): compute settlement. Knowing the compressible thickness, H_i, to be 32 - 8 or 24 cm, the total settlement, S, finally can be computed by use of equation 14,

$$S = \frac{e_i - e_f}{1 + e_i} H_i = \frac{2.41 - 2.08}{1 + 2.41} 24 \text{ cm} = 2.3 \text{ cm}.$$

Time rate -- The preceding settlement analysis indicated that the load in the problem will very slowly sink about 2.3 cm deeper than 8 cm as a result of the gradual consolidation of the sedimentary stratum upon which it initially rested.

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[5] The average initial void ratio also can be obtained another way. At 8 cm, the sedimentary overburden stress is 3.4 g/cm^2, at 20 cm (1/2 layer thickness), 9.2 g/cm^2; and 32 cm, 15.3 g/cm^2 (Fig. 25). These stresses correspond to void ratios of 2.53, 2.32, and 2.21 (Fig. 10); and e_i is computed to be 2.35 by averaging these numbers. Using this value in equation 14 settlement is computed to be 1.9 cm.

TABLE 8. STRUCTURAL LOAD COMPUTATION FOR CORE A 31

Mass of load = 5 kg; dimensions of footing: 10 cm x 10 cm, area = 100 cm^2, $x = \frac{10}{2}$ and $y = \frac{10}{2}$;

sediment pressure (stress) $= \frac{5000}{100} = 50\ g/cm^2$.

Depth Below 8 cm, z (cm)	Depth Below Top of Core (cm)	$m = \frac{x}{z}$ & $n = \frac{y}{z}$ (Fig. 26)	m = n (Fig. 26)	Ratio p_z/p, from Fig. 26	p_z (g/cm^2)	Total Pressure at Center of Load ($4\ p_z$) (g/cm^2)	Natural Overburden Stress from Fig. 25-A (g/cm^2)	Load Plus Overburden Stress (Plotted as B, Fig. 25) (g/cm^2)
0	8	5/0	infin.	.25	12.5	50.0	3.4	53.4
10	18	5/10	.50	.085	4.25	17.0	8.3	25.3
15	23	5/15	.33	.044	2.20	8.8	10.7	19.5
20	28	5/20	.25	.028	1.40	5.6	13.3	18.9
25	32	5/25	.20	.018	.90	3.6	15.3	18.9
30	38	5/30	.17	.013	.65	2.6	18.7	21.3
40	48	5/40	.12	.006	.30	1.2	24.3	25.5
50	58	5/50	.10	.005	.25	1.0	30.2	31.2

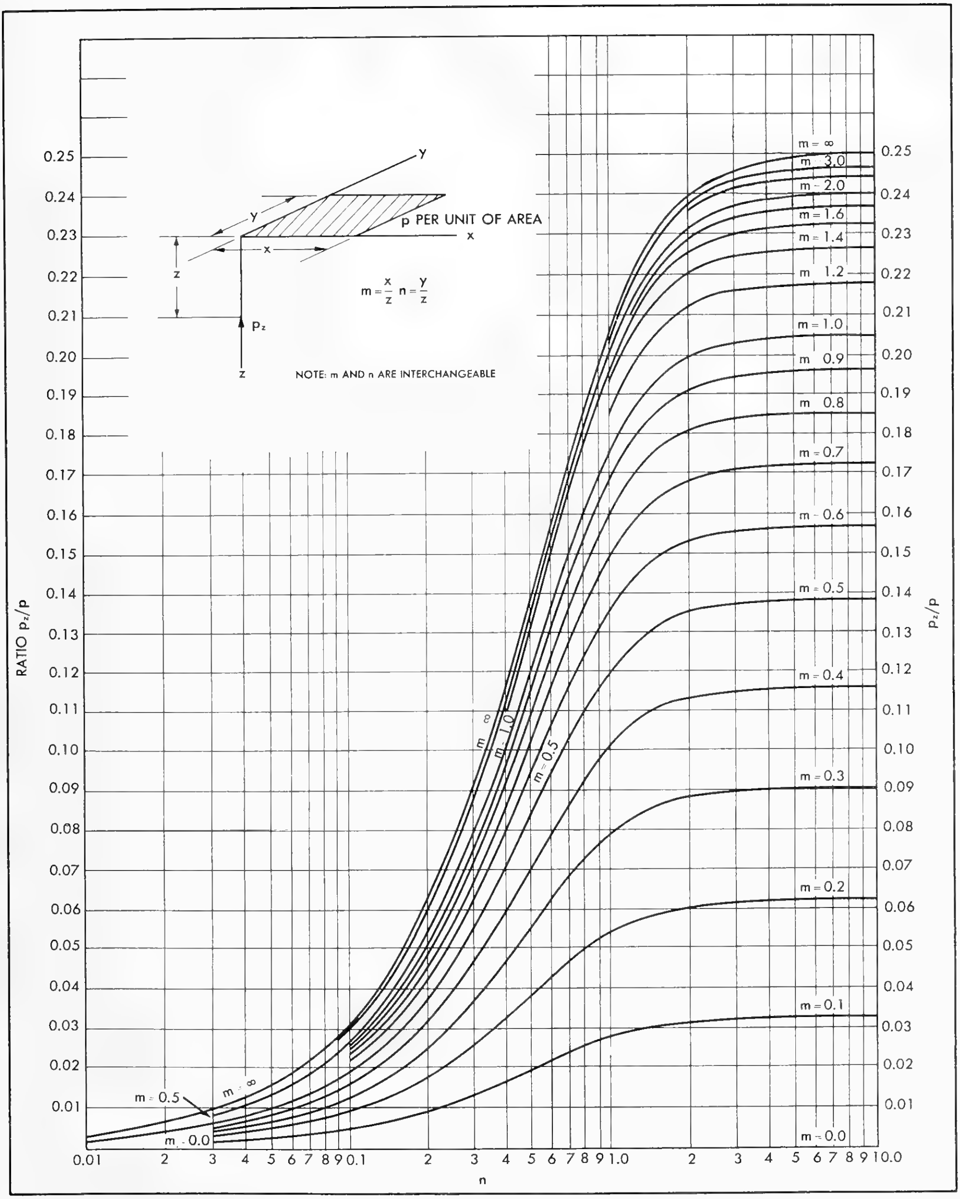

FIGURE 26. GRAPH FOR FINDING RATIO p_z/p FOR A RECTANGULAR AREA UNIFORMLY LOADED. (After Palmer, 1953, p. 39)

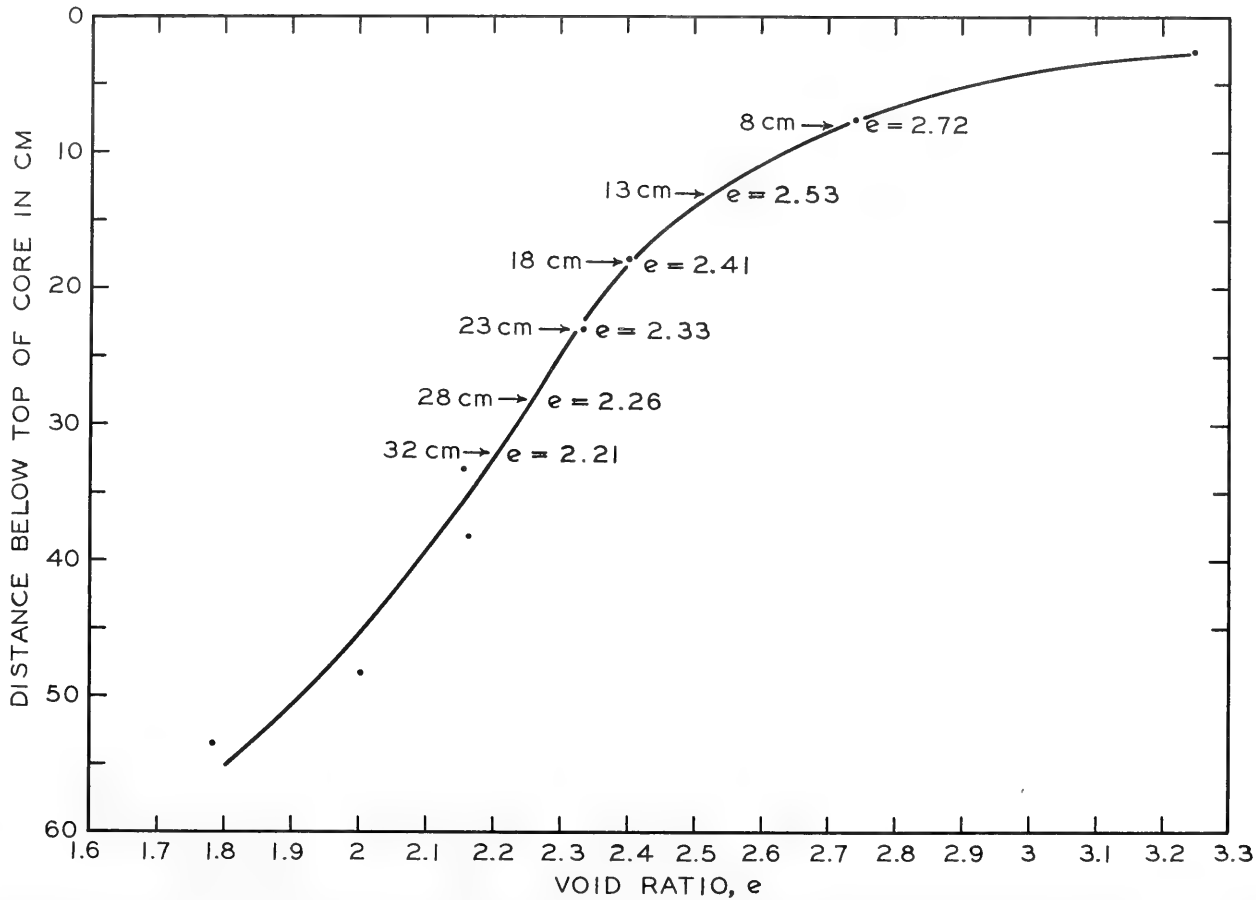

FIGURE 27. DEPTH RELATED TO MEASURED VOID RATIO (100% SATURATION) FOR CORE A 31

TABLE 9. COMPUTATION OF e_i AND FINAL STRESS FOR DETERMINATION OF e_f

Depth[1/] (cm)	e (from Fig. 27)	Average Stress (from Fig. 25, Curve B) (g/cm^2)
8	2.72	53.4
13	2.53	38.2
18	2.41	25.3
23	2.33	19.5
28	2.26	18.9
32	2.21	18.9
	14.46	174.2

average $e_i = \frac{14.46}{6} = 2.41$ average final stress $= \frac{174.3}{6} = 29.0\ g/cm^2$

The amount of time for this consolidation to occur can be computed from the formula[6]

$$t_{(years)}\ \frac{N\ H^2}{1.3 \times 10^6\ c_v} \qquad (15)$$

where the drainage takes place in one direction only (up), t is the time in years, N is a factor corresponding to the percentage completion of consolidation in any given problem, H is the thickness in cm of the stratum undergoing consolidation, and c_v is the coefficient of consolidation in cm^2/minute obtained from a laboratory time consolidation relationship. This relationship is

$$c_v = \frac{0.2\ H_{50}{}^2}{4\ t_{50}} \qquad (17)$$

where H_{50} is the thickness in cm at the time (t_{50}) in minutes of completion of 50 percent of primary consolidation.

[1/] Depths selected at 5 cm intervals.

[6] This formula usually is given as $t_{(years)}\ \frac{N\ H^2}{1400\ c_v}$ where H is in feet. (16)

Continuing our problem, we shall seek the time required for 50 percent consolidation to occur in the layer 8 to 32 cm. H is equal to 24 cm. The ratio, U, of the initial stress at the surface to the initial stress at the bottom of the compressible layer must be computed before the N factor can be determined. These figures are obtainable from curve B in Figure 25: at 8 cm, 53.4 g/cm^2 and at 32 cm, 18.9 g/cm^2. Thus, U is 53.4/18.9 or 2.8. N is found to be about 0.35 from Figure 28 (after Palmer, 1953, p. 50).

At the average void ratio of 2.41 (Table 9), the extrapolated c_v from Figure 10 is about 0.01 cm^2/minute. This computation is

$$t_{50} = \frac{N\ H^2}{1.3 \times 10^6\ c_v} = \frac{0.35\ (24)^2}{1.3 \times 10^6\ (0.01)} = 0.016 \text{ years}$$

The time required for nearly 100 percent consolidation to occur (N = 5, from Figure 28) is about 0.22 years.

In conclusion, one should keep in mind the admonition of Taylor (1948, p. 300) that "settlement analyses usually give results which at best are crude estimates."

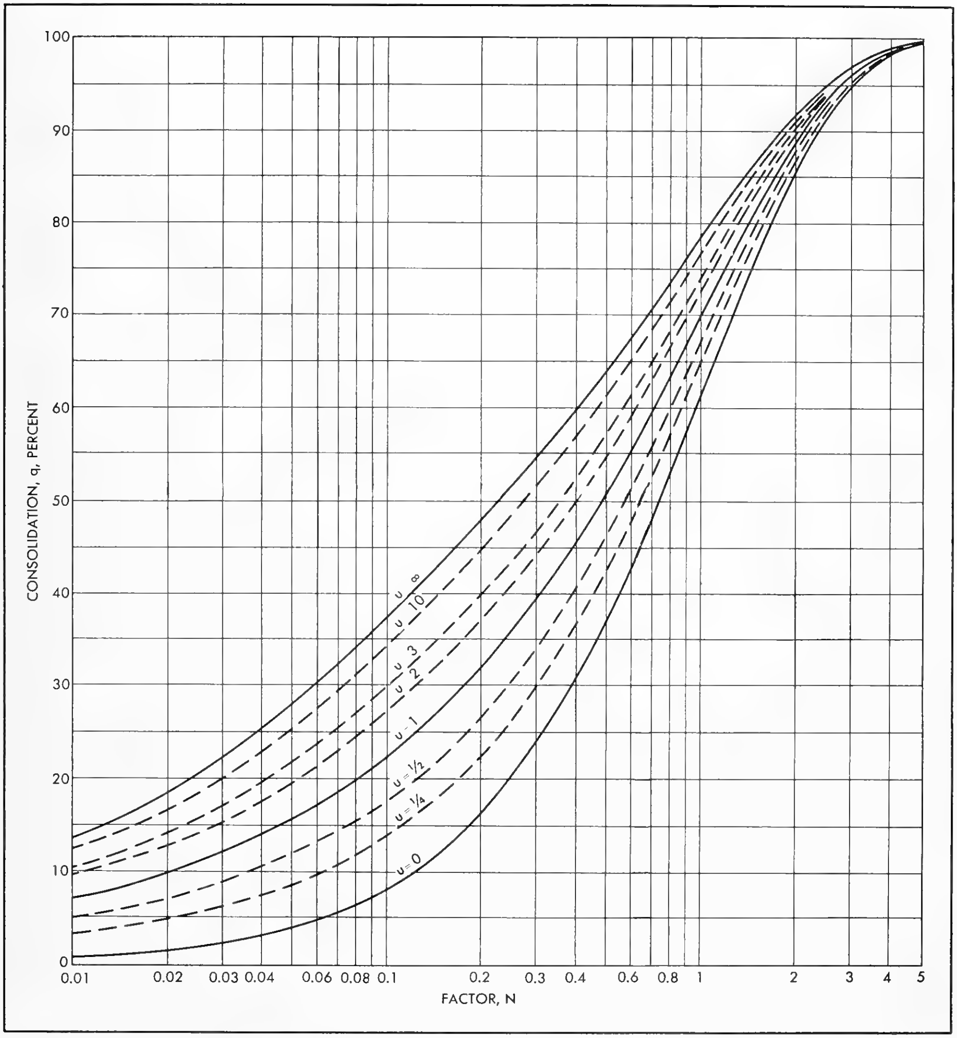

FIGURE 28. GRAPH FOR FINDING N FACTORS. (After Palmer, 1953, p. 50)

VI. SUMMARY AND CONCLUSIONS

Thirty gravity and piston cores were collected from eight areas in the North Atlantic, West Mediterranean, and Central Pacific. They were transported to the laboratory in Washington, D. C., with protection against mechanical vibration and, so far as possible, in an upright position. Sediments were composed of clayey silt- and silty clay-sized particles, predominantly of terriginous origin.

A knowledge of the gross core recovery ratio (gross core length/corer penetration distance) is of importance for engineering and other investigations requiring sample depth. Evidence is presented from recent Swedish investigations that this ratio may not be always 100 percent for piston cores, as is commonly assumed. The ratio is variable for gravity-type cores; it appears to be a function of the design. Well-engineered gravity corers have gross recovery ratios about 100 percent in the upper 40 to 75 cm and smaller ratios below this depth. Poorly-engineered (for undisturbed sampling) gravity corers may have core shortening proportional to the distance penetrated and gross recovery ratios much less than 100 percent.

All gravity and piston cores in general use by oceanographers appear to take disturbed samples; disturbance is worse in some than in others. Better designed samplers or in-place tests, perhaps by neutron probe or from deep submersibles, are required for more valid strength measurements.

Sediments described in this report are considered to be disturbed, the principal cause of which is sample collection because proper care was exercised elsewhere to protect against major disturbance. A quantitative figure for the reduction of in-place strength is unknown, but the amount is believed sufficiently small to allow the use of the strength values reported for most engineering purposes.

Shear strength, expressed as cohesion, was measured in the laboratory by compression and vane tests that are briefly described. Most cores were continuously sampled and measured for cohesion every 5 or 10 cm.

Cohesion, graphically related to depth for each core, usually increases with increasing depth. This increase in some instances is regular and in others highly irregular. A few cores possessed a relatively uniform strength distribution from top to bottom. The least cohesion measured was 4.2 g/cm^2 and the greatest 234 g/cm^2. Most cores showed minimum cohesion at the top and maximum values at intermediate depths.

Sediment sensitivity ranged from 1.6 to 26, with most values falling between 2 and 6, medium to very sensitive. It is speculated that in-place sensitivity in many

instances may be greater than determined in the laboratory because of sampler disturbance to the core. Total strength regain by thixotropy is considered unlikely.

A substantial portion of the report is devoted to the practical application of shear strength and laboratory-determined consolidation data to the solution of sea-floor sediment ultimate bearing capacity and consolidation problems, examples of which are given.

REFERENCES

American Society of Civil Engineers and American Society for Testing Materials Committee (1958) Tentative definitions of terms and symbols relating to soil mechanics. Book of ASTM standards including tentatives, part 4. Philadelphia, American Society for Testing Materials, p. 1211-1228.

Arrhenius, G. (1952) Sediment cores from the east Pacific. Reports of the Swedish Deep-sea Expedition 1947 - 1948, vol. 5, fasc. 1, 227 p.

Bishop, A. W. and Henkel, D. J. (1957) The measurement of soil properties in the triaxial test. London, Edward Arnold, Ltd., 190 p.

Bjerrum, L. (1954a) Geotechnical properties of Norwegian marine clays. Géotechnique, vol. 4, p. 49-69.

- - - (1954b) Theoretical and experimental investigations on the shear strength of soils. Norwegian Geotechnical Institute Publication No. 5, 113 p.

- - - and Kjaernsli, B. (1957) Analysis of the stability of some Norwegian natural clay slopes. Geotéchnique, vol. 7, p. 1-16.

Burmister, D. M. (1946) Aims and objectives for A.S.T.M. Committee D-18 on soils. ASTM (American Society for Testing Materials) Bulletin No. 142, p. 38-40.

Butcher, W. S. (1951) Bearing strength of fine-grained sediments. Navy Electronics Laboratory - National Research Council Cooperative Research Group, NEL:NRC:CRG:18, 2 p.

Capper, P. L. and Cassie, W. F. (1960) The mechanics of engineering soils. 3rd ed. London, E. & F. N. Spon, 315 p.

Coulomb, C. A. (1776) Essai sur une application des règles de maximis et minimis à quelques problemes de statique, relatifs à l'architecture. Mémoires de Mathématique et de Physique, L'Académie Royale des Sciences, Paris, p. 343-382.

Emery, K. O. and Dietz, R. S. (1941) Gravity coring instrument and mechanics of sediment coring. Geological Society of America Bulletin, vol. 52, p. 1685-1714.

Ericson, D. B. and Wollin, G. (1956) Correlation of six cores from the equatorial Atlantic and the Caribbean. Deep-sea Research, vol. 3, p. 104-125.

- - - Ewing, M., Wollin, G., and Heezen, B. C. (1961) Atlantic deep-sea sediment cores. Geological Society of America Bulletin, vol. 72, p. 193-286.

Ewing, M. (1960) The ice ages - theory. Alberta Society of Petroleum Geologists Journal, vol. 8, p. 191-201.

Hamilton, E. L. (1960) Ocean basin ages and amounts of original sediments. Journal of Sedimentary Petrology, vol. 30, p. 370-379.

Heezen, B. C. (1952) Discussion to methods of exploring the ocean floor by R. S. Dietz, p. 200-205 in oceanographic instrumentation. National Academy of Sciences - National Research Council Publication 309, 233 p.

Hough, B. K. (1957) Basic soils engineering. New York, Ronald Press, 513 p.

Hvorslev, M. J. (1936) Conditions of failure for remolded cohesive soils. International Conference on Soil Mechanics and Foundation Engineering. Proceedings. 1st - Cambridge, Mass., vol. 3, p. 51-53.

- - - (1937) Über die Festigkeitseigenschaften gestörter bindiger Böden. Ingeniørvidenskabelige Skrifter, A no. 45, (Danmarks Naturvidenskabelige Samfund) Copenhagen, 159 p.

- - - (1949) Subsurface exploration and sampling of soils for civil engineering purposes. Vicksburg, Mississippi (U. S. Army, Corps of Engineers) Waterways Experiment Station, 521 p.

- - - and Stetson, H. C. (1946) Free-fall coring tube; a new gravity bottom sampler. Geological Society of America Bulletin, vol. 57, p. 935-950.

Jakobson, B. (1954) Influence of sampler type and testing method on shear strength of clay samples. Royal Swedish Geotechnical Institute Proceedings, No. 8, 59 p.

Jürgenson, L. (1934) The shearing resistance of soils. Boston Society of Civil Engineers Journal, vol. 21, p. 242-275. Reprinted in: Contributions to soil mechanics 1925 - 1940. Boston Society of Civil Engineers, 1940, p. 184-217.

Kallstenius, T. (1958) Mechanical disturbances in clay samples taken with piston samplers. Royal Swedish Geotechnical Institute Proceedings, No. 16, 75 p.

Keller, G., Richards, A. F., and Recknagel, J. H. (in press) Prevention of water loss through CAB sediment core liners. Deep-sea Research.

Kullenberg, B. (1947) The piston core sampler. Svenska Hydrografisk-Biologiska Kommissionens Skrifter, Tredje serien: Hydrografi, Band 1, Häfte 2, p. 1-46.

- - - (1955) Deep-sea coring. Reports of the Swedish Deep-sea Expedition, vol. 4, fasc. 1, p. 35-96.

Lambe, T. W. (1951) Soil testing for engineers. New York, Wiley, 165 p.

- - - (1961) A mechanistic picture of shear strength in clay, p. 555-580 in Research conference on shear strength of cohesive soils. New York, American Society of Civil Engineers.

Meyerhof, G. G. (1951) The ultimate bearing capacity of foundations. Géotechnique, vol. 2, p. 301-332.

Moore, D. G. (1959) Stability of deep-sea sedimentary slopes, p. 474-476 in International Oceanographic Congress preprints. Washington, American Association for the Advancement of Science, 1022 p.

- - - (in press) Submarine slumps. Journal of Sedimentary Petrology.

- - - and Richards, A. F. (submitted) Conversion of "relative shear strength" measurements by Arrhenius on east Pacific cores to conventional units of shear strength. Géotechnique.

Newmark, N. M. (1935) Simplified computation of vertical pressures in elastic foundations. University of Illinois Bulletin 33, Engineering Experiment Station Circular No. 24, 19 p.

Osterberg, J. O. (1957) Introduction, p. 1-7 in Symposium on vane shear testing of soils. American Society for Testing Materials Special Technical Publication No. 193, 70 p.

Phleger, F. B and Parker, F. L. (1951) Ecology of Foraminifera, northwest Gulf of Mexico. Geological Society of America Memoir 46, 152 p.

Piggot, C. S. (1941) Factors involved in submarine core sampling. Geological Society of America Bulletin, vol. 52, p. 1513-1524.

Prandtl, L. (1920) Über die Härte plastischer Körper. Koniglichen Gesellschaft der Wissenschaften zu Göttingen, Mathematisch - physikalische Klasse, p. 74-85.

Pratje, O. (1934) Sind die Bodenprofile aus den Röhrenloten ohne Unterbrechungen? Annalen Hydrographie und Maritimen Meteorologie, vol. 62, p. 137-144.

Richards, A. F. (1960) Plastic-barrel corer and its use in studies of engineering and mass physical properties of deep-sea sediments. Association d'Oceanographie Physique, Union Géodésique et Geophysique Internationale, General Assembly at Helsinki, July-August 1960, Abstracts of Communications, p. 154-155.

- - - (in preparation) Investigations of deep-sea sediment cores, 2. Mass physical properties. U. S. Hydrographic Office Technical Report No. 106.

- - - and Keller, G. H. (in press) Plastic-barrel sediment corer. Deep-sea Research.

Rosenqvist, I. Th. (1953) Considerations on the sensitivity of Norwegian quick-clays. Gēotechnique, vol. 3, p. 195-200.

Shepard, F. P. (1954) Nomenclature based on sand-silt-clay ratios. Journal of Sedimentary Petrology, vol. 24, p. 151-158.

Skempton, A. W. (1942) An investigation of the bearing capacity of a soft clay soil. Institution of Civil Engineers Journal, No. 7, 1941-42, p. 307-321.

- - - (1951) The bearing capacity of clays. Building Research Congress Proceedings 1951, Division 1, part 3, p. 180-189.

- - - (1953) Soil mechanics in relation to geology. Yorkshire Geological Society Proceedings, vol. 29, p. 33-62.

Skempton, A. W. (1960) Terzaghi's discovery of effective stress, p. 42-53 in From theory to practice in soil mechanics. New York, Wiley, 425 p.

- - - and Bishop, A. W. (1950) The measurement of the shear strength of soils. Géotechnique, vol. 2, p. 90-108.

- - - and Northey, R. D. (1952) The sensitivity of clays. Géotechnique, vol. 3, p. 30-53.

Soil Mechanics and Foundation Division Committee on Glossary of Terms and Definitions (1958) Glossary of terms and definitions in soil mechanics. American Society of Civil Engineers Proceedings vol. 84: Soil Mechanics and Foundations Journal (Proc. Paper 1826) SM4 43 p.

Spangler, M. G. (1960) Soil engineering. 2nd ed., Scranton, Pennsylvania, International Textbook, 483 p.

Sykes, L. R. (1960) An experimental study of compressional velocities in deep sea sediments. B. S. & M. S. Thesis, Massachusetts Institute of Technology, 74 p.

Taylor, D. W. (1948) Fundamentals of soil mechanics. New York, Wiley, 700 p.

Terzaghi, C. (1925) Principles of soil mechanics: VIII Future development and problems. Engineering News-record, vol. 95, p. 1064-1068.

Terzaghi, K. (1943) Theoretical soil mechanics. New York, Wiley, 510 p.

- - - (1944) Ends and means in soil mechanics. Engineering Journal (Canada), vol. 27, p. 608-615.

- - - (1955) Influence of geological factors on the engineering properties of sediments. Economic Geology, 50th Anniversary Volume 1905 - 1955, p. 557-618.

- - - and Peck, R. B. (1948) Soil mechanics in engineering practice. New York, Wiley, 566 p.

Tschebotarioff, G. P. (1951) Soil mechanics, foundations, and earth structures. New York, McGraw-Hill, 655 p.

Whitman, R. V. (1961) Some considerations and data regarding the shear strength of clays, p. 581-614 in Research conference on shear strength of cohesive soils. New York, American Society of Civil Engineers.

U. S. Hydrographic Office (1955) Instruction manual for oceanographic observations. Hydrographic Office Publication No. 607, 2nd ed., 210 p.

Vold, R. C. (1956) Opptagning av uforstyrrede jordprøver. Teknisk Ukeblad, No. 8: reprinted in Norwegian Geotechnical Institute Publication No. 17, p. 1-14.

APPENDIX

OUTLINE OF SHEAR STRENGTH TEST PROCEDURE

I. Compression test[7]

A. Test

1. Take initial extensometer dial reading
2. Apply first load increment and start test timing.
3. Record dial reading at 15 seconds.
4. Record dial reading and add next load increment at 30 seconds.
5. Repeat steps 3 and 4.

B. Computations

1. Determine initial height of sediment cylinder.
2. Determine change of height (Δh) at 30-second readings.
3. Determine percent strain ($\Delta h/h$) at 30 second intervals.
4. Determine unit load/original area (total load/A).
5. Determine corrected unit load, which equals unit load x (1-strain).
6. Plot strain versus corrected unit load (Fig. 7a)

II. Vane Shear Test[8]

A. Test

1. Secure sample container to prevent its rotation during the test.
2. Lower vane into sample until top of vane is at least 0.75 inches below the surface.
3. Record initial dial settings (outer dial shows degrees of vane rotation and inner dial shows degrees of applied torque - see Fig. 8b).
4. Start test by turning on motor.

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[7] Prepared by Mr. C. M. Yeomans.

[8] From information prepared by Mr. G. H. Keller.

5. If results are to be plotted, record dial settings every 2° of vane rotation.

6. After shear occurs, the vane will rotate at the same speed as the application of torque (6°/min); the inner dial will maintain the reading at the time of shear failure, although the outer dial will continue to advance. Five consecutive vane rotation readings that are similar should be recorded, in addition to noting a vane rotation of at least 20°, for a valid test.

B. Computations

1. Compute difference between initial and final torque dial settings to obtain ΔT.

2. Compute shear strength from the formula:

$$s = \frac{F\ \Delta T}{2\pi\ r^2\ (h + 0.667r)} = \frac{F\ \Delta T}{0.360} \qquad (18)$$

where s is the shear strength, F is the spring factor[9], ΔT is the amount of torque in degrees required to produce shear failure, r is the vane radius (0.25 in), and h is the vane height (0.75 in).

3. If the results are to be plotted, compute the shear stress for every 2° of vane rotation (ΔT will be the amount of torque required to produce deformation at the given vane rotation).

4. Plot vane shear strength versus vane rotation (Fig. 7b).

[9] The spring used for these tests was specially constructed by the manufacturer; it has a factor of about 0.013 (for one spring: 0.01197 from 0° to 58° and 0.01344 from 58° to 187° of torque).